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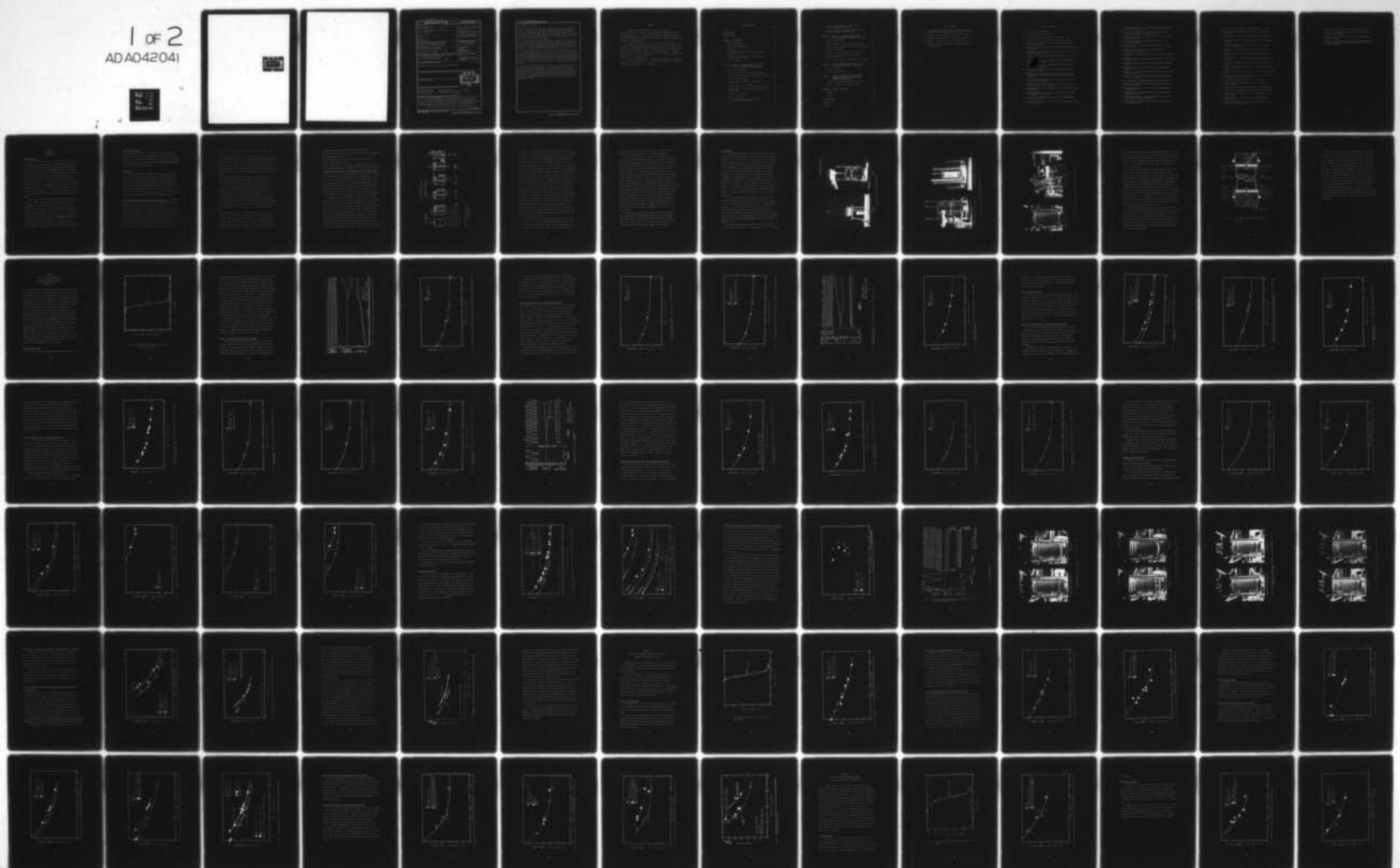

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The study reported herein is a continuation of an earlier preliminary study on the effect of frictionless ends on the cyclic strength of sands. The earlier study used laboratory data from static tests to develop a working hypothesis relating the effect of end restraint on dilation tendency, which in turn influenced the undrained static and the cyclic strength of soil.

The study described herein continues with a detailed investigation of

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20. ABSTRACT (Continued).

the effect of frictionless ends versus regular ends in the cyclic triaxial testing of various soils. The study involved performing some additional 130 cyclic triaxial tests on samples of the following soils, using both regular and frictionless ends: Monterey No. 0 sand, Sacramento River sand, L. A. Harbor sand, and undisturbed and remolded Champlain clay. These tests were performed under different conditions of density, anisotropic consolidation stress ratio, and cyclic frequency.

Since the frictionless ends used in this study for the sands had short prongs in the centers to keep the samples from sliding off to the side, the possible effect of these prongs on the cyclic strength had to be defined. Therefore, a series of comparative tests were performed for each sand soil studied using regular ends with and without prongs. It was observed that prongs had no effect on the cyclic strength of these sand soils.

In order to assure that the viscous grease used to lubricate the ends was in fact acting as a lubricant, the cyclic tests had to be performed much slower than commonly done for earthquake studies. Therefore, the question of the effect of cyclic frequency on the indicated cyclic strength of the soil also had to be considered. For this purpose, tests were performed on samples with regular and with frictionless ends at cyclic frequencies of 1.0 Hz and 0.05 Hz. From the results of these tests, it was observed that the frequency at which the tests were run did not affect the cyclic strength of the soils tested.

Thus, after establishing that the prongs and the frequency did not influence the cyclic strength of these soils, comparative tests were run to determine the effect of end restraint. The results of the end restraint studies showed that for relatively clean sands there was an increase of 10 to 30 percent in cyclic strength by using frictionless ends rather than regular ends. The silty sands and clays tested were not influenced by the type of end restraint. Furthermore, the data showed a recognizable relation of an increasing strengthening influence of frictionless ends with an increasing tendency for dilation.

PREFACE

This report was prepared by Mr. Frank J. Vernese and Professor Kenneth L. Lee under Contract DACW-39-75-M-4712 as part of ongoing work at the U. S. Army Engineer Waterways Experiment Station (WES) under CWIS 31145 work unit entitled "The Liquefaction Potential of Earth Dams and Foundations."

The work was directed by Dr. W. F. Marcuson III, Research Civil Engineer, Earthquake Engineering and Vibrations Division (EE&VD), Soils and Pavements Laboratory (S&PL). General guidance was provided by the following S&PL personnel: Messrs. J. P. Sale, Chief, S. J. Johnson, Special Assistant, and W. C. Sherman and Dr. F. G. McLean, former Chief and Chief, EE&VD, respectively.

Directors of WES during this study and the preparation and publication of this report were COL G. H. Hilt, CE, and COL J. L. Cannon, CE. Technical Director was Mr. F. R. Brown.

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CHAPTER 1

INTRODUCTION

Purpose and Scope

Within the past few years the effect of end restrain in the triaxial test has become of interest to researchers and engineers. A previous report by the second named author presents a literature review of published data, which compares the results of regular and frictionless ends for static tests. In addition, a limited number of cyclic test results on one fine sand are also presented. As the present study was being concluded, a few additional comparative cyclic triaxial tests on a fine sand were reported by Mulilis (7). These two limited sets of data are insufficient to define the effect of frictionless ends on a general basis. Therefore, the study reported herein was undertaken to obtain data from a wider variety of soils and different test conditions than included in the earlier reports.

In his earlier report, Lee (2) had observed that undrained static strength, dilatency and end restrain were interrelated, and as a working hypothesis, suggested that a similar tendency might also apply to cyclic strengths. Thus, the present study was planned to include among other things, soils which exhibited a wide range of dilatent tendencies under static loading. In all, some 130 cyclic triaxial tests were performed. Thus, by combining the results of these tests with the logic and data in the earlier report (2), a deeper understanding of the effect of end restraint on the cyclic triaxial strength of soils has been achieved and is presented herein.

Sources of Information

Background information on the effect of end restraint on the static triaxial strength of soils and the cyclic strength of loose Sacramento River sand was obtained from the Lee report (2). The cyclic loading data presented in this report were obtained by direct testing in the UCLA soil mechanics laboratory.

Background

Although the effect of end restraint on the strength of soil has just recently come into consideration for the cyclic triaxial test, there have been many previous studies made on the influence of end conditions in the static triaxial testing of materials. The results from these earlier studies are summarized in the previous report (2) and are not repeated herein. However, for the sake of clarity, a summary of the conclusions from static tests with frictionless ends and the working hypothesis developed therefrom are presented below.

Conclusions From Static Tests with Frictionless Platens. The following conclusions were made by Lee (2) on the results of available data from many studies with static loading tests on soils:

(1) Using regular ends or rough platens in a triaxial test yields a non-uniform normal and shear stress and strain distribution, with most of the strain occuring in the middle rather than near the ends of the sample. This phenomena is observed in most triaxial tests, where, upon application of sufficient stress the sample proceeds to bulge in the center while the ends remain fixed. To reduce this effect on the measured strength to within tolerable limits, samples with

regular ends are conventionally run with a height to diameter ratio, $h/d = 2$.

(2) Using regular ends with $h/d = 2$ tends to accentuate non-uniformities in the sample which may result in one or more major shear planes developing prematurely. On the other hand, with frictionless ends the sample has a uniform stress and strain distribution and no premature shear plan development. Thus, tests can be conducted on short samples ($h/d = 1$) with frictionless ends to obtain accurate stress, strain and strength results.

(3) Since volume changes are dependent on shear strains, the non-uniform shear strain distribution caused by regular ends will have an effect on the volumetric strains. In addition, there are dead zones near the rough ends which do not undergo large shear strains and, therefore do not undergo as strong dilatancy tendencies as the zones of major shear strains in the middle of the sample. With the use of frictionless ends, a more uniform strain distribution is achieved and thus the overall dilatent volumetric strain tendencies are greater for free ends than for samples with regular ends. Differences in volume change tendency leads to higher critical confining pressures (σ_{3crit}) using samples with frictionless ends, than for samples with regular ends.

(4) In the undrained test, the soil sample behavior is directly analogous to the behavior in the drained tests. Instead of measuring volumetric changes, in the undrained tests the pore pressure change is recorded. But unlike volume changes, which are localized and cannot spread readily, pore pressure changes can spread throughout the sample as fast as some small amount of water can flow. Therefore, pore

pressure equalization is achieved throughout the sample.

(5) The use of regular ends in the static undrained tests overestimates ϕ' by approximately 1 degree.

(6) Using regular ends in undrained static tests underestimates σ_{3crit} by approximately 3 kg/cm^2.

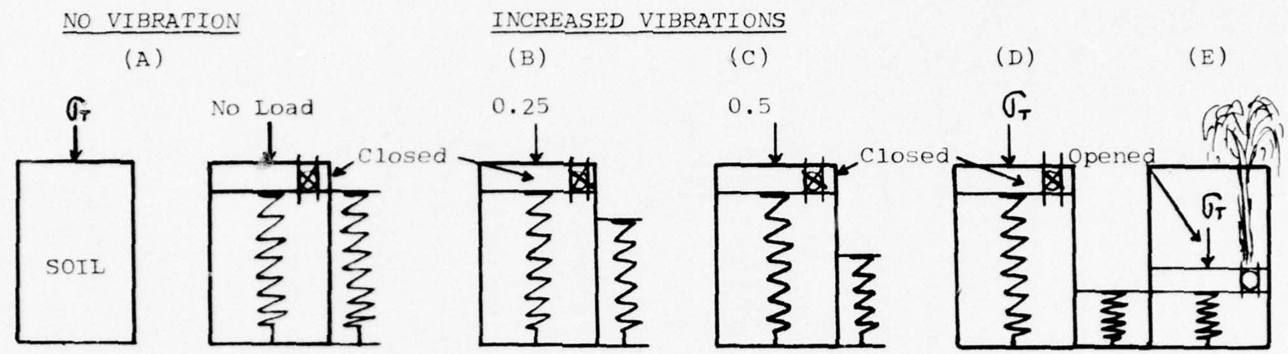
(7) The combined effect of regular ends on σ_{3crit} and ϕ' leads to lower undrained strengths for regular ends, than for frictionless ends.

Prediction of the Effect of End Restraint in Cyclic Triaxial Tests.

It can be seen that from the conclusions on the effect of end restraint of the static triaxial test given above, some predictions can be made as to the effect of end restraint in the cyclic triaxial test. The data presented by Lee (2) showed that saturated samples with frictionless ends have greater cyclic strength because the overall dilatency effect is greater than with regular ends. As a background to the cyclic strength effect, it is appropriate to explain the basic cause of liquefaction in a saturated sand under cyclic loading conditions.

When a saturated sand is vibrated, it tends to compact and decrease in volume. However, the volume can decrease only if the drainage is allowed to occur. If the drainage is prevented, the tendency to decrease in volume will result in an increase in the pore water pressure. This effect can best be visualized in the common mechanical analogy of the 'piston and the spring'. Figure 1-1 shows a spring assumed to be immersed in a water tight cylinder filled with water. The spring represents the compressible soil skeleton of a mass of saturated soil with a total overburden pressure of (σ_T). The water in the cyliner represents the water in the voids. The piston is provided with a stopcock which is assumed to be closed so that no water

FIGURE 1-1
PISTON AND SPRING ANALOGY



	(A)	(B)	(C)	(D)	(E)
Load Carried by Water =	0	0.25 σ_T	0.5 σ_T	σ_T	0
Load Carried by Spring=	0	0	0	0	σ_T
Percent Consolidation =	0	0	0	0	100
Effective Stress(σ') =	σ_T	σ_T-0.25σ_T	σ_T-0.5 σ_T	σ_T-σ_T=0	σ_T
Soil Condition * =	A	B	B	C	D

*Note: A = Insitu
B = No Liquefaction
C = Liquefaction
D = Settlement

can escape. Figure 1-1a shows the system at rest with no induced vibrations. When vibrations are induced, the spring tends to compress according to the superimposed loads shown by the springs outside the cylinders. Under the load of $0.25\sigma_T$, the spring would tend to compress as shown in Figure 1-1b. However, it cannot compress unless the piston descends, and the piston cannot descend because the water cannot escape. Therefore, the water must carry all of the superimposed load and the effective stress is reduced by $0.25\,\sigma_T$. With increasing vibrations, the spring tends to compress as shown in Figures 1-1c and 1d; again the spring cannot compress since there is no drainage. Thus, unless the stopcock is opened as in Figure 1-1e, the load carried by the water will continue to increase until it builds up to the point where it equals the total load σ_T. When this occurs, the spring does not carry any of the super-imposed load and the effective stress becomes zero. At this point, if the spring were to be replaced by the compressible soil skeleton of a mass of saturated soil and the water in the cylinder replaced by the water in the soil voids, the soil would lose its strength completely and develop a liquefied state. Thus, when a saturated soil is vibrated and drainage is prevented, the tendency to decrease in volume will result in an increase in the pore water pressure, and, if the pore water pressure builds up to a point where it equals the overburden pressure, the effective stress becomes zero and liquefaction occurs.

From this definition, it is evident that the liquefaction potential or the cyclic strength is directly dependent on the rate of pore pressures built up during cyclic loading. The liquefaction potential is also dependent on the amount of dilation that occurs per cycle of

stress because as the pore pressure decreases, the effective stress increases and, thus, the sample's cyclic strength increases.

In the cyclic triaxial test, as the load is applied, the pore pressure increases and decreases according to the direction of the load (extension or compression). But on each cycle, the net residual pore pressure is higher than at the end of the previous cycle. This process continues until the peak pore pressure during a cycle equals the confining pressure. At this point, initial liquefaction has been reached. After the sample has reached initial liquefaction, the nature of the pore pressure response changes. The highest pore pressure occurs when the axial stress is zero. Since this high value is equal to or close to the confining pressure, the effective stress is close to zero at that stage. However, as the load increases, the sample tends to dilate and the pore pressure decreases, which increases the effective stress and strength enough to carry the axial load on the next cycle. In this process, the sample strains until stabilized by dilation, then the sample strains at a slower rate. Thus, the greater the tendency for the sample to dilate, the greater the number of cycles required to cause a certain specified amount of strain. In this report, failure has been defined at 5 percent double amplitude strain, unless otherwise stated.

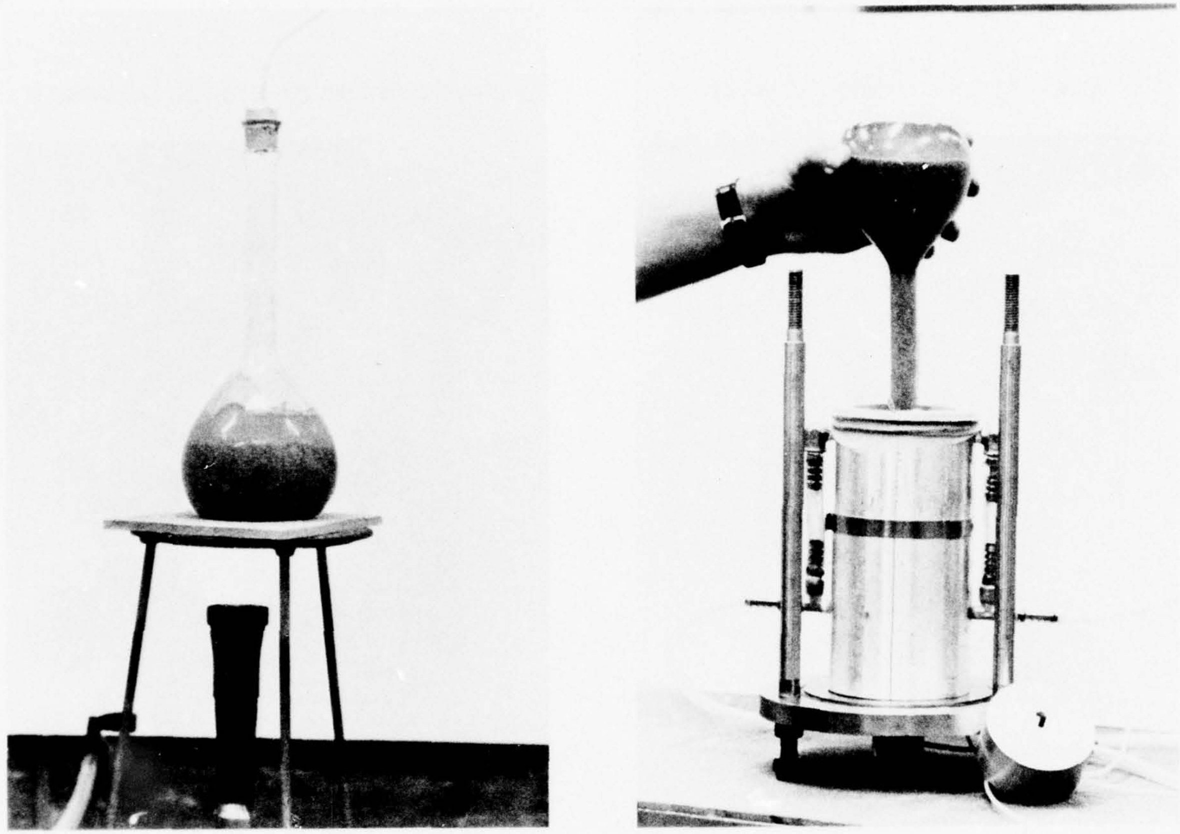
In summary, the available data suggests as a working hypothesis, that since samples with frictionless ends dilate more than samples with regular ends, it follows that samples with frictionless ends in the cyclic triaxial test will strain less per cycle and thus require more cycles to reach "failure" than samples with regular ends.

Testing Program

Soil liquefaction studies involving cyclic triaxial tests use isotropically consolidated samples. Seismic slope stability studies require testing of anisotropically consolidated soil. In specifying the stress condition required for each type of testing, it has been found convenient to use the terms anisotropic consolidation stress ratio $Kc = \sigma_{1c}/\sigma_{3c}$. The terms σ_{1c} and σ_{3c} are the effective major and minor principal stress during consolidation. Isotropic consolidation is used to simulate in the laboratory triaxial test the appropriate field effective vertical stresses on a horizontal plane under a level surface. This condition corresponds to Kc = 1.0. Anisotropic consolidation is used to simulate in the laboratory, stresses in the field under sloping surfaces. Typical test conditions are Kc = 1.5 or Kc = 2.0. To make this study of frictionless ends complete, some test series were performed for all three Kc ratio conditions.

The soils tested in this report include: Monterey No. 0 Sand at relative densities (Dr) of 60, 80 and 90 percent and Kc ratios of 1.0, 1.5 and 2.0; Sacramento River Sand at Dr = 38 and 80 percent and Kc = 1.0; Los Angeles Harbor Sand at Dr = 60 percent and Kc = 1.0; and Champlain Clay using both undisturbed and remolded specimens. Cyclic triaxial tests were conducted for both regular and frictionless ends on each soil. For convenient reference, all pertinent data regarding these tests have been summarized and are presented in Table A-1 (Appendix - A).

All sand samples were prepared using the wet raining method as shown in Figures 1-2 a through f. First, a sample of soil was weighed

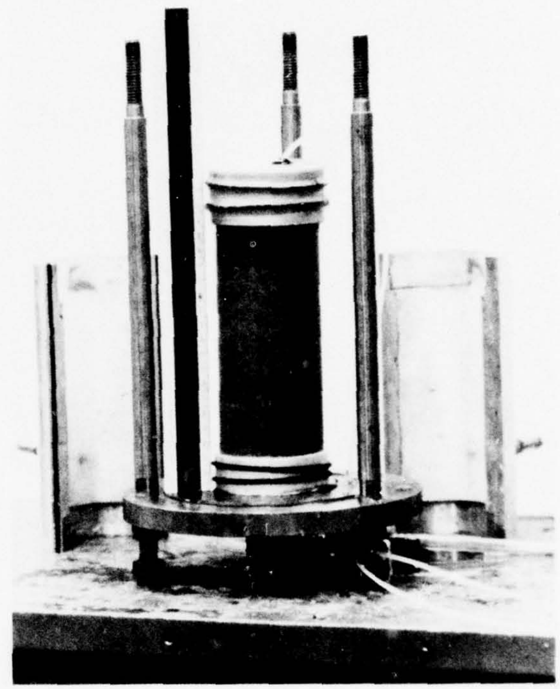


BOILING
(A)

RAINING
(B)

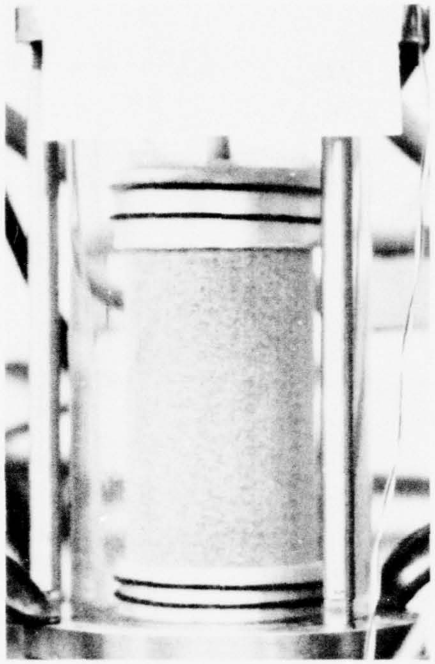
FIG. 1-2A,2B SEQUENCE OF SAMLPE PREPARATION

VIBRATING SAMPLE TO CORRECT HEIGHT
(C)

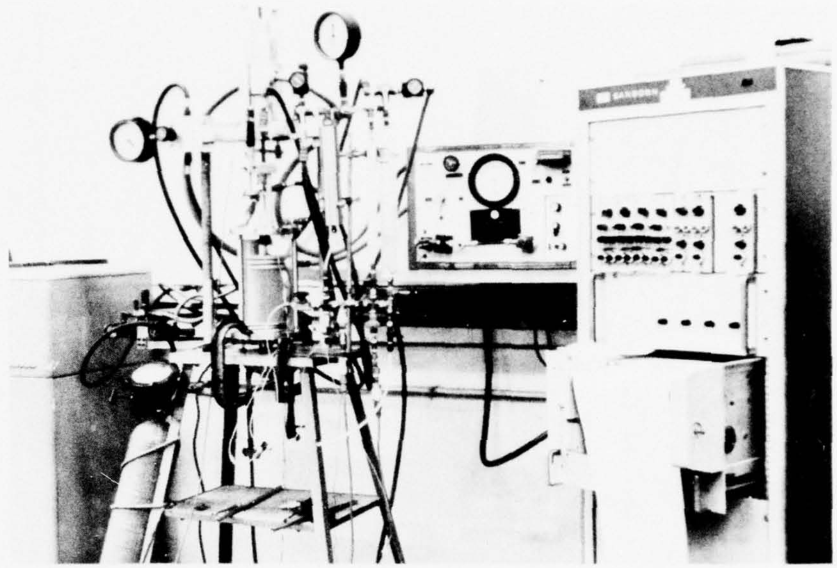


FORMED SAMPLE & SPLIT MOLD
(D)

FIGS. 1-2C,2D SEQUENCE OF SAMPLE PEPARATION (Cont.)



SAMPLE IN TRIAXIAL CELL
(E)



SAMPLE IN TESTING APPARATUS
(F)

FIGS. 1-2E,2F SEQUENCE OF SAMPLE PREPARATION (Cont.)

to the correct density and boiled in de-aired water to saturate. Then the soil was rained into a mold containing de-aired water. The sides of the mold were vibrated to bring the soil to the correct height and density. This same procedure was used for each sand sample tested with both frictionless and regular ends. The only deviation was in the type of end restraint.

Frictionless ends were made by using two layers of rubber, each 0.012 in. thick and each separated by a generous smear of high vacuum silicone grease. In addition, 1/8 in. diameter by 0.5 in. long prongs were fixed in the center of the cap and base. The purpose of the prongs was to keep the sample from sliding off the ends while at the same time allow for drainage and pore pressure measurements. A diagram of a sample with frictionless ends is shown on Fig. 1-3. This set-up was essentially the same as used by Lee (2). Since frictionless ends have short prongs in the centers to keep the samples from sliding off the ends and to aid in drainage, the effect of these prongs on the cyclic strength had to be established. This was accomplished by running two series of tests; one series used regular ends with prongs and the other used regular ends without prongs.

In addition to establishing the effect of prongs on the cyclic strength, the amount of lubrication which could be developed by running tests at the conventional relatively fast frequency of 1.0 Hz had to be delineated. It was felt that this short frequency would not be sufficient to allow the viscous grease to act as a lubricant. Thus, tests were conducted for each soil with regular and frictionless ends at cyclic frequencies of both 0.05 Hz (1 cycle per 20 seconds) and 1.0 Hz (1 cycle per second).

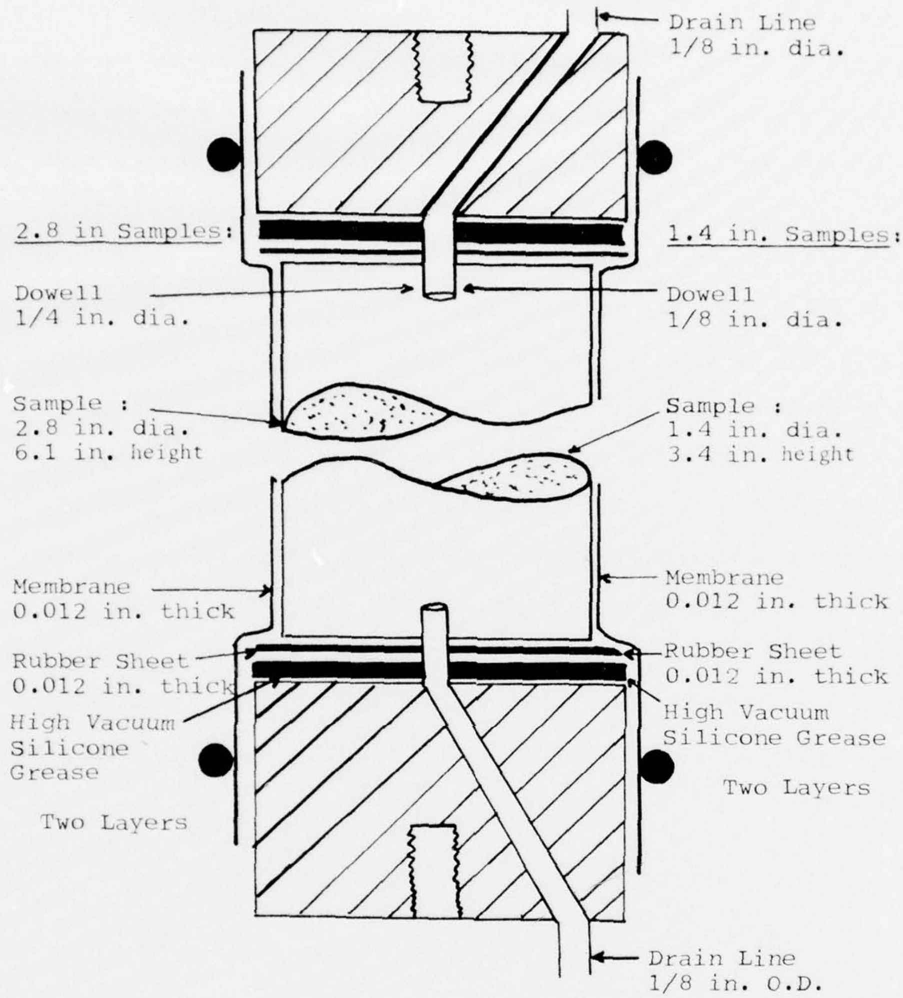


FIG. 1-3 FRICTIONLESS CAPS AND BASES USED IN THIS STUDY

A summary of the relevant conditions pertaining to each cyclic triaxial test is presented in the Appendix. Failure in the cyclic triaxial tests was defined by the criterion, the number of cycles N of uniform axial stress σ_{dp} required to cause a certain specified axial strain ε_p. Unless otherwise specified, the failure strain criteria for isotropically consolidated samples was ε_p = 5 percent double amplitude (peak to peak) regardless of whether or not the strains were completely symmetrical (extension = compression). This criteria was approximately the same as initial liquefaction (pore pressure = cell pressure and effective stress = zero) for many cases. For anisotropically consolidated samples, strains always developed more in the compression direction than in extension and the excess pore pressures may never have reached the cell pressure to give a classical liquefaction (zero effective stress) condition. Therefore, following conventional practice, failure was defined in terms of the cyclic conditions which caused 5 percent accumulative axial compression strain.

CHAPTER 2

EFFECT OF END RESTRAINT ON THE CYCLIC TRIAXIAL STRENGTH OF MONTEREY SAND

Monterey Sand is a clean uniform sand mined from the beach at Monterey, California and separated by washing and sieving into various size fractions. A grain size distribution curve for the No. 0 sand used in this report is shown in Figure 2-1. The particle sizes range between 0.2 mm and 1.0 mm with D50 = 0.36 mm. The maximum and minimum void ratios are 0.85 and 0.56 respectively. Numerous cyclic triaxial tests were performed on this soil at three relative densities; Dr = 60, 80 and 90 percent. The samples were prepared in the same manner as described in Chapter 1 and were tested using both regular and frictionless ends at cyclic frequencies of 1.0 Hz and 0.05 Hz. The first series of tests were performed on 1.4 in. diameter samples, 3.4 in. in height with an effective isotropic consolidation pressure of 14.5 psi. The second series of tests were conducted on 2.8 in. diameter samples, 6.1 in. in height with an effective consolidation pressure of σ_{3c} = 14.5 psi and K_c = 1.0, 1.5 and 2.0. From the results of these tests, it was possible to study the effects of; prongs, frequency, sample size, and end restraint on the cyclic strength of this Monterey Sand. Additional cyclic strength data for this soil is contained in a comprehensive study by Mulilis (7).

Behavior of Loose Sand

The results of a typical cyclic triaxial test, using regular ends,

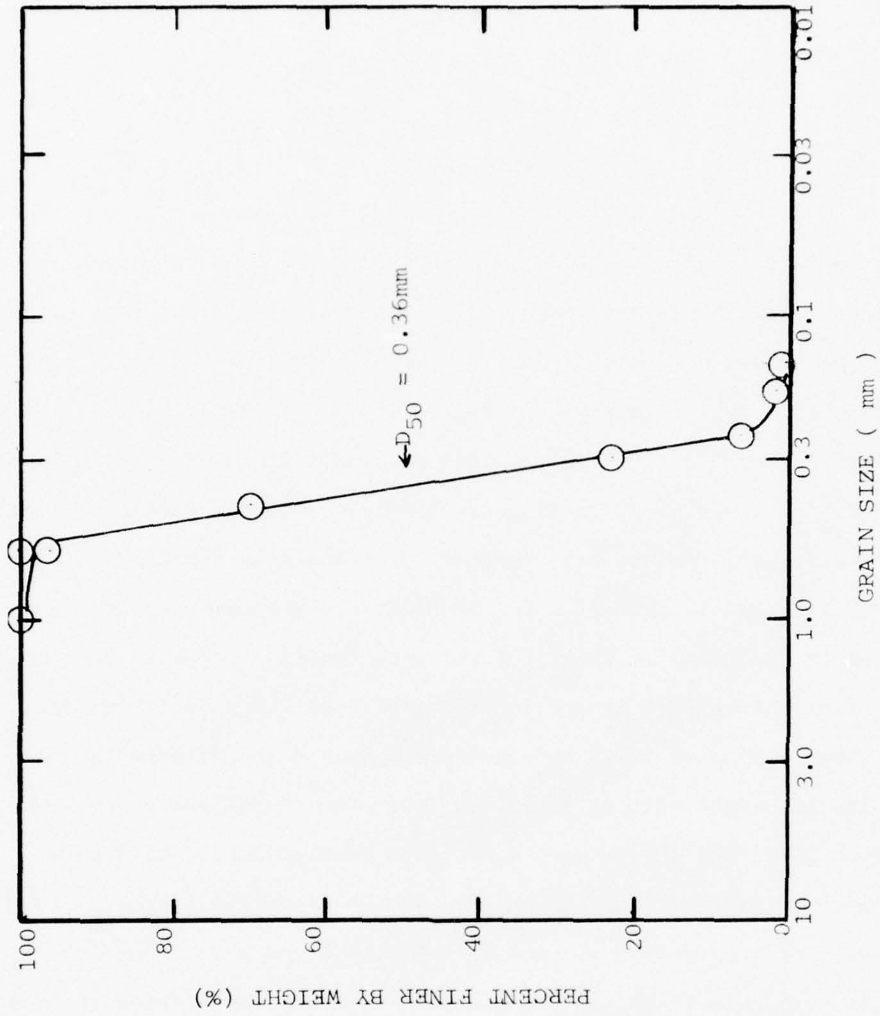


FIG. 2-1 GRAIN SIZE DISTRIBUTION CURVE FOR MONTEREY NO. 0 SAND

on loose sand (Dr = 60 percent) are shown on Figure 2-2. In this test, a cyclic deviator stress, of constant amplitude σ_{dp} = 5.6 psi was applied to the sample under an effective isotropic confining pressure of 14.5 psi. The resulting changes in axial strain and pore pressure were recorded as shown. There was no significand changes in the axial strain until the pore pressure reached a point close to the confining pressure. In fact, for 50 cycles, there was no observable deformation at the level of sensitivity of the recorder. Then within the next 9 cycles, the failure strain (5 percent double amplitude) was reached. In 5 more cycles the axial strain reached 10 percent double amplitude. It should be noted that for 1.4 in. samples, the peak axial load starts reducing as soon as the sample exhibits significant strain, of about 2 percent. However, this occurred for all 1.4 in. samples tested and thus, the results should be comparable from sample to sample.

The relationship between the cyclic stress and the number of cycles to cause failure for samples run at the same density and confining pressure, but by different deviator stresses is shown in Figure 2-3. It should be noted that for the same confining pressure and sample density, the number of stress cycles required to cause failure decreases as the cyclic deviator stress is increased.

Effect of Prongs on the Strength of Loose Sand

The frictionless caps and bases used in this study had prongs in their centers to keep the sample from sliding off and to aid in drainage. The regular samples had no prongs. Therefore, the effect of prongs on the cyclic strength had to be established. This was accomplished by inserting prongs in regular ends and comparing the

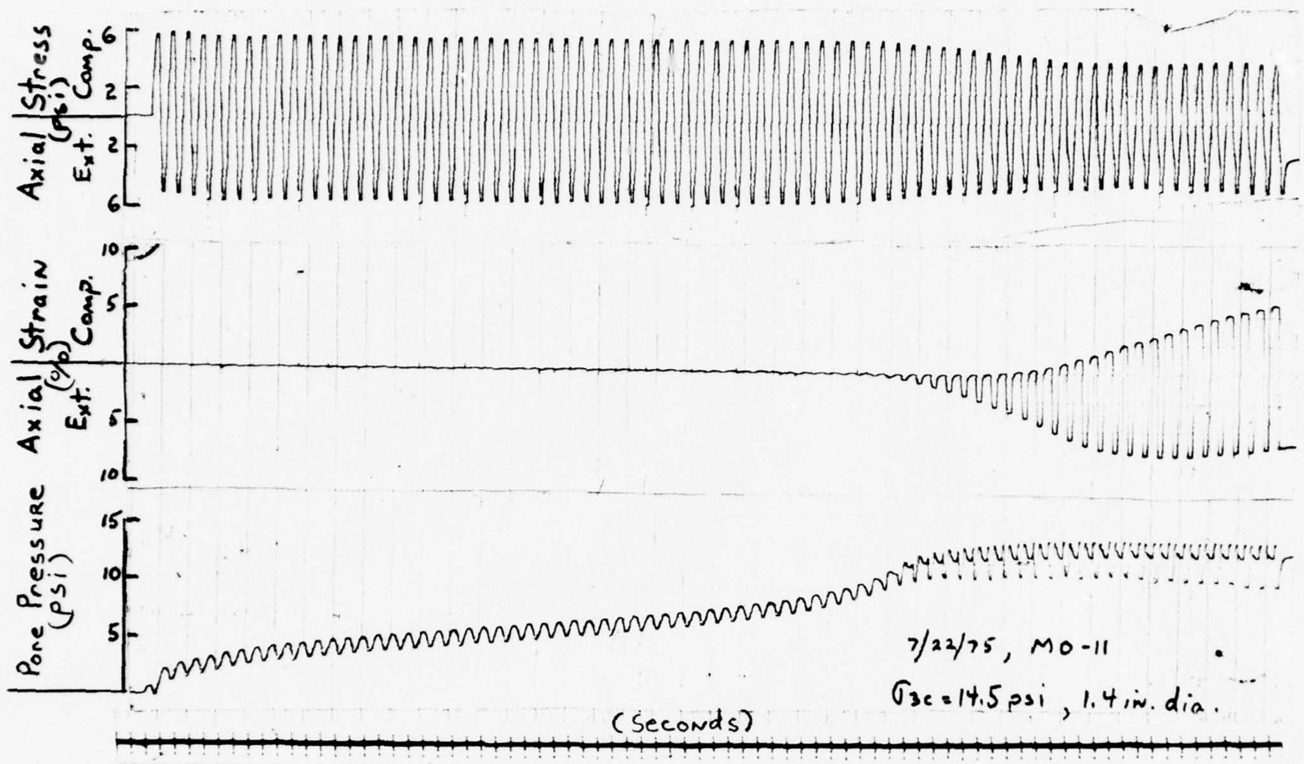


FIG.2-2 RECORD OF A TYPICAL CYCLIC TRIAXIAL TEST ON LOOSE MONTEREY SAND,REGULAR ENDS

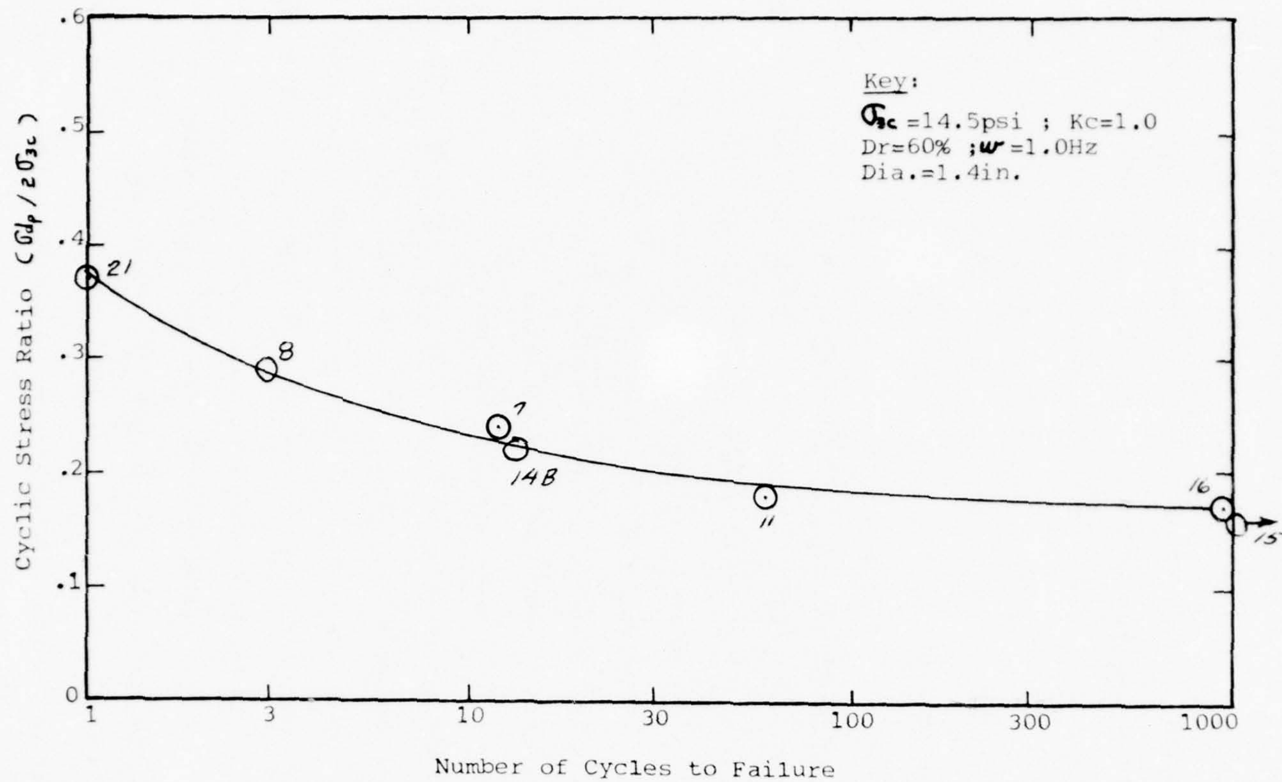


FIG. 2-3 STRENGTH DATA FROM CYCLIC TRIAXIAL TESTS ON LOOSE MONTEREY SAND -- REGULAR ENDS

results of these tests with tests using regular ends without prongs.

Strength data from cyclic triaxial tests on loose sand for samples with regular ends with prongs is shown in Figure 2-4. These samples were prepared in the same manner and run at the same density and confining pressure as the samples with regular ends without prongs. The data from both sets of tests are shown on Figure 2-5. Comparing these results, shows that the prongs have no effect on the cyclic strength of loose Monterey Sand.

Effect of End Restraint on the Strength of Loose Sand

The results of a typical cyclic triaxial test using frictionless ends on a 1.4 in. diameter sample of loose sand are shown in Figure 2-6. In this test, a cyclic deviator stress of constant amplitude σ_{dp} = 6.0 psi was applied to the sample previously consolidated under an effective isotropic confining pressure σ_{3c} = 14.5 psi. Note that even though this sample had a higher cyclic stress than the sample with regular ends previously shown, it took more cycles to reach failure. In comparing the two records, it is observed that the frictionless sample, shown in Figure 2-6, required about 20 cycles after the first significant strain occured to reach failure (5 percent double amplitude); whereas, the sample with regular ends, shown in Figure 2-2, needed only about 10 cycles to reach the same cyclic strain condition. Also, it took an additional 30 cycles to reach 10 percent strain while the sample with regular ends needed only 5 additional cycles.

The relationship between the cyclic stress and the number of cycles to cause failure for frictionless samples run under the same conditions. but using different cyclic deviator stresses is shown in Figure 2-7.

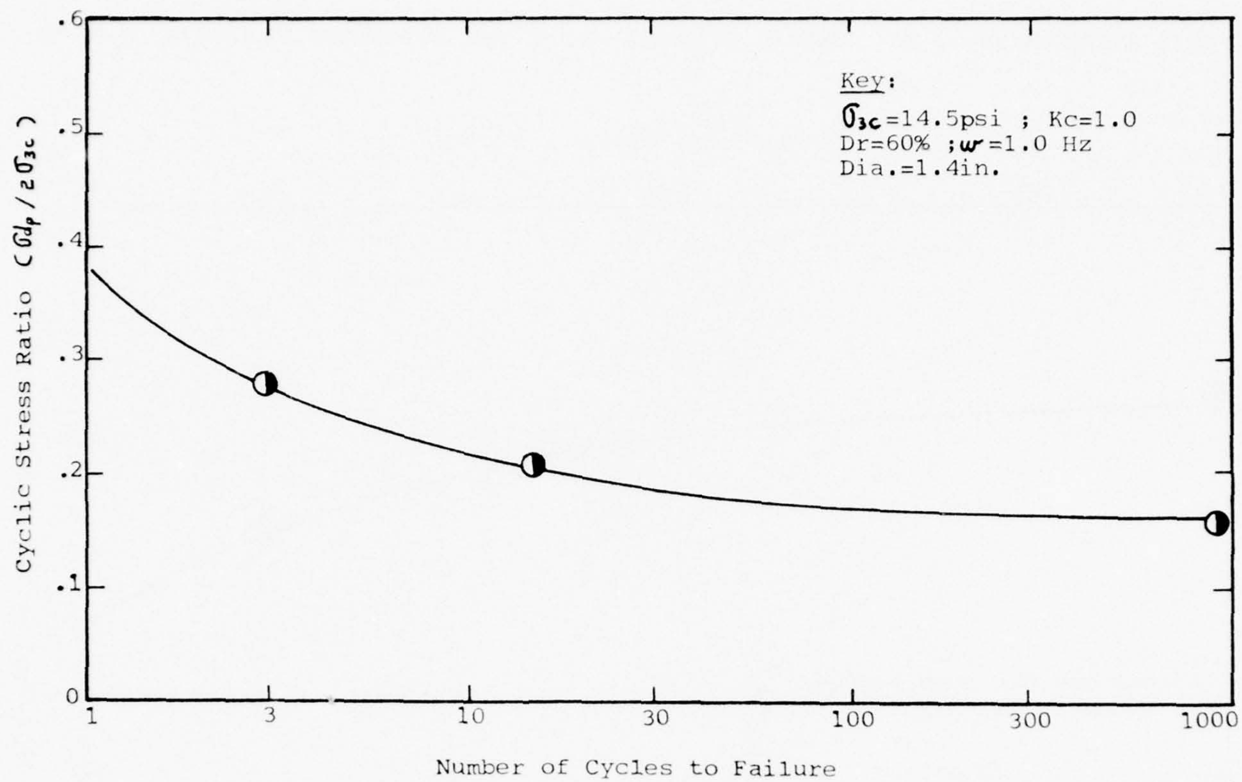


FIG. 2-4 STRENGTH DATA FROM CYCLIC TRIAXIAL TESTS ON LOOSE MONTEREY SAND - REGULAR ENDS WITH PRONGS

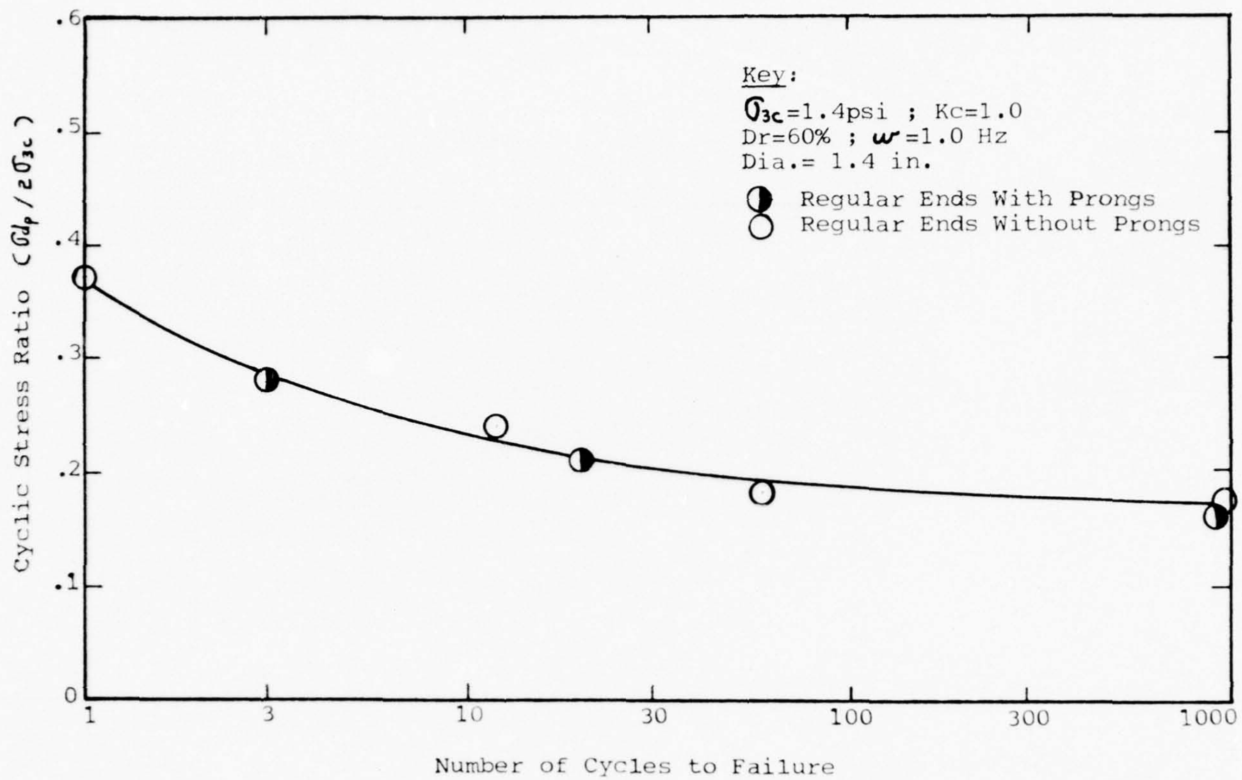


FIG. 2-5 EFFECT OF PRONGS ON THE CYCLIC TRIAXIAL STRENGTH OF LOOSE MONTEREY SAND

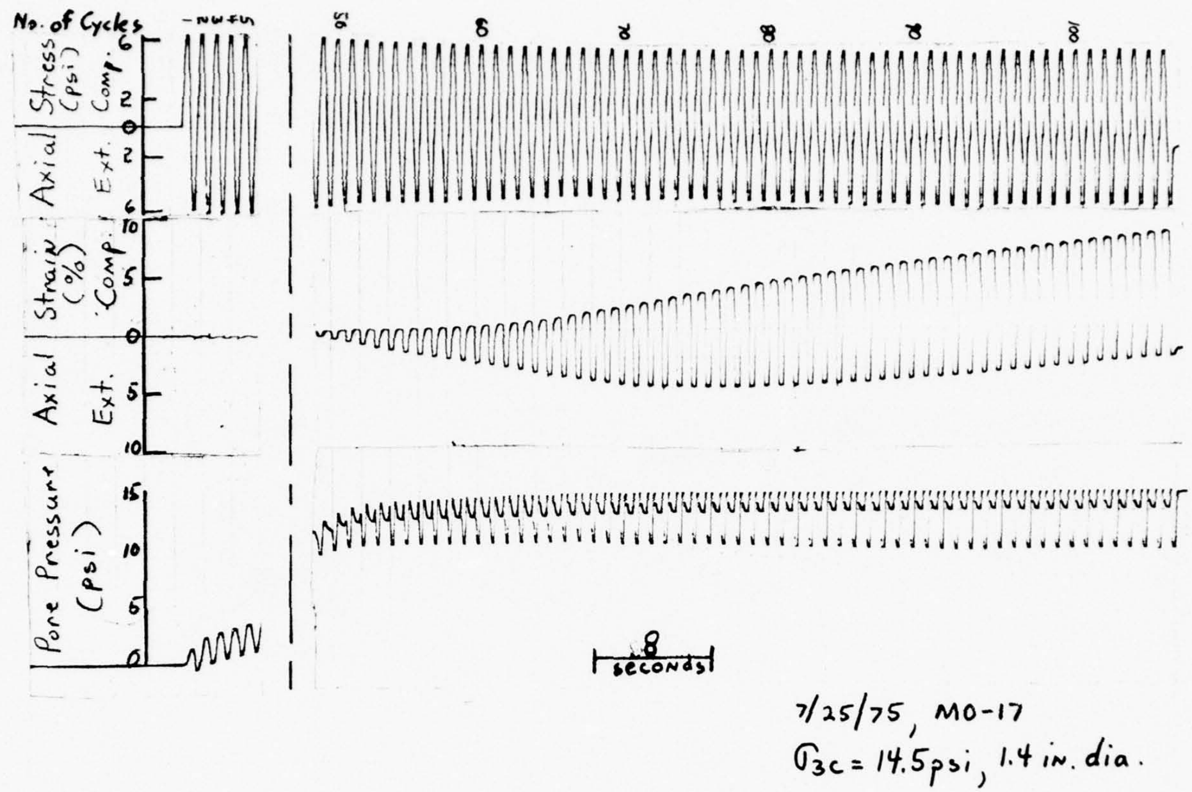


FIG. 2-6 RECORD OF A TYPICAL CYCLIC TRIAXIAL TEST ON LOOSE MONTEREY SAND, FRISTIONLESS ENDS

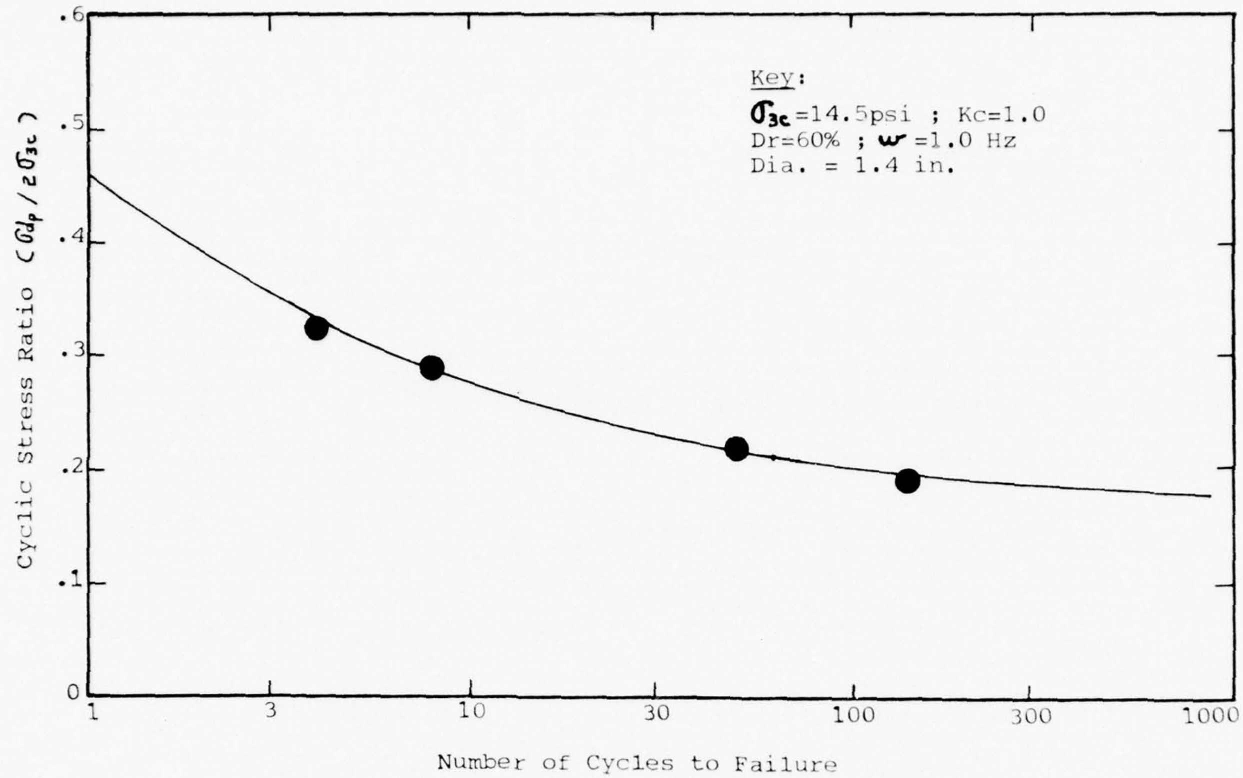


FIG. 2-7 STRENGTH DATA FROM CYCLIC TRIAXIAL TESTS ON LOOSE MONTEREY SAND - FRICTIONLESS ENDS

A comparison of this strength data with that shown in Figure 2-3, for regular ends, is presented on Figure 2-8. The samples tested with frictionless ends are approximately 10 to 20 percent stronger than those tested with regular ends.

Effect of Rubber Thickness

In addition to tests run with two layers of grease, a series of tests were conducted using only one layer of rubber and grease. These tests were performed to see if there was any measurable effect of the thickness of the end rubber and grease on the cyclic triaxial strength. A comparison of the results between samples run with one and two layers of grease is presented in Figure 2-9. From these results it seems that the difference in strength is insignificant. However, the data are limited to only three tests. More test data are needed to make definitive conclusions.

Effect of Cyclic Frequency on the Strength of Loose Sand

Up to this point, all tests (both with regular and frictionless ends) were conducted at a frequency of 1.0 Hz (1 cycle per second). However, in running frictionless tests, there is some question as to the amount of lubrication that is developed at 1.0 Hz frequency. Thus, additional tests were performed using a cyclic frequency of 0.05 Hz (1 cycle per 20 seconds), for both frictionless and regular end samples.

Strength data from cyclic triaxial tests on loose sand for frictionless ends run at 0.05 Hz is shown on Figure 2-10. Comparing this data with the results shown on Figure 2-6, for frictionless ends run at

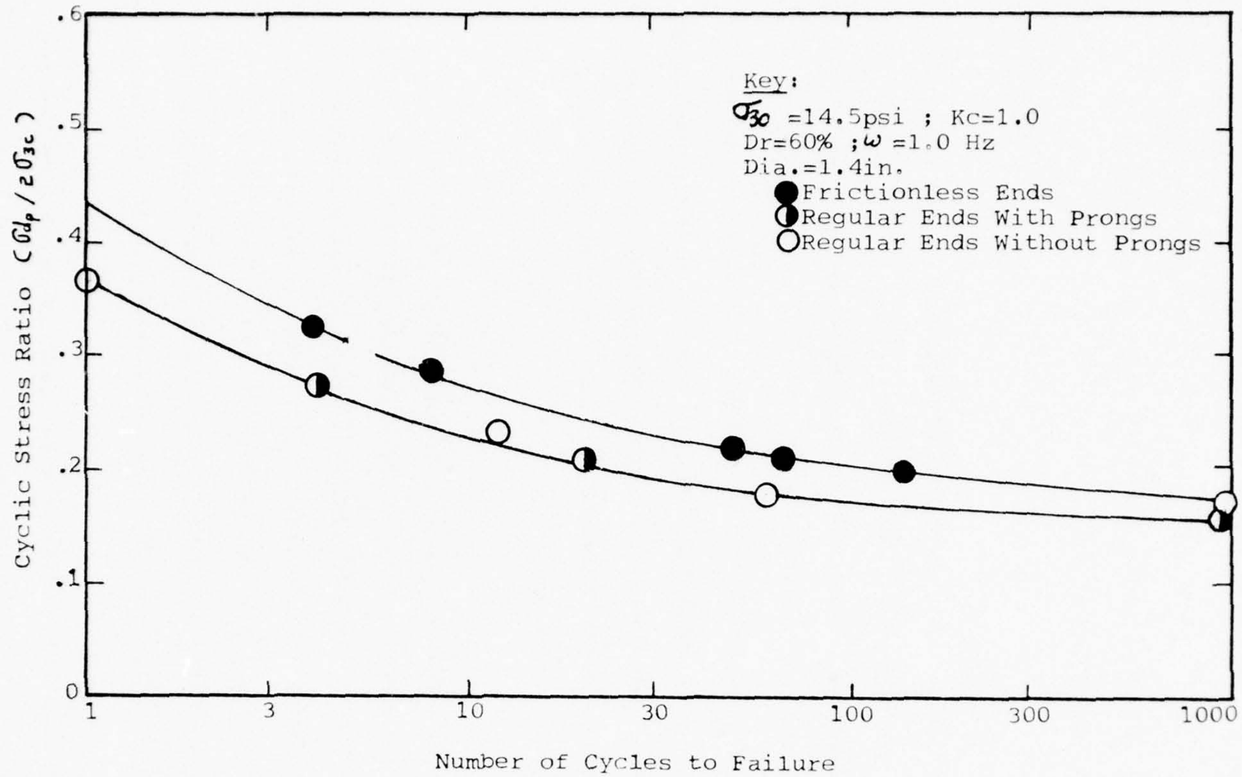


FIG. 2-8 COMPARISON OF STRENGTH DATA FROM CYCLIC TRIAXIAL TESTS ON LOOSE MONTEREY SAND

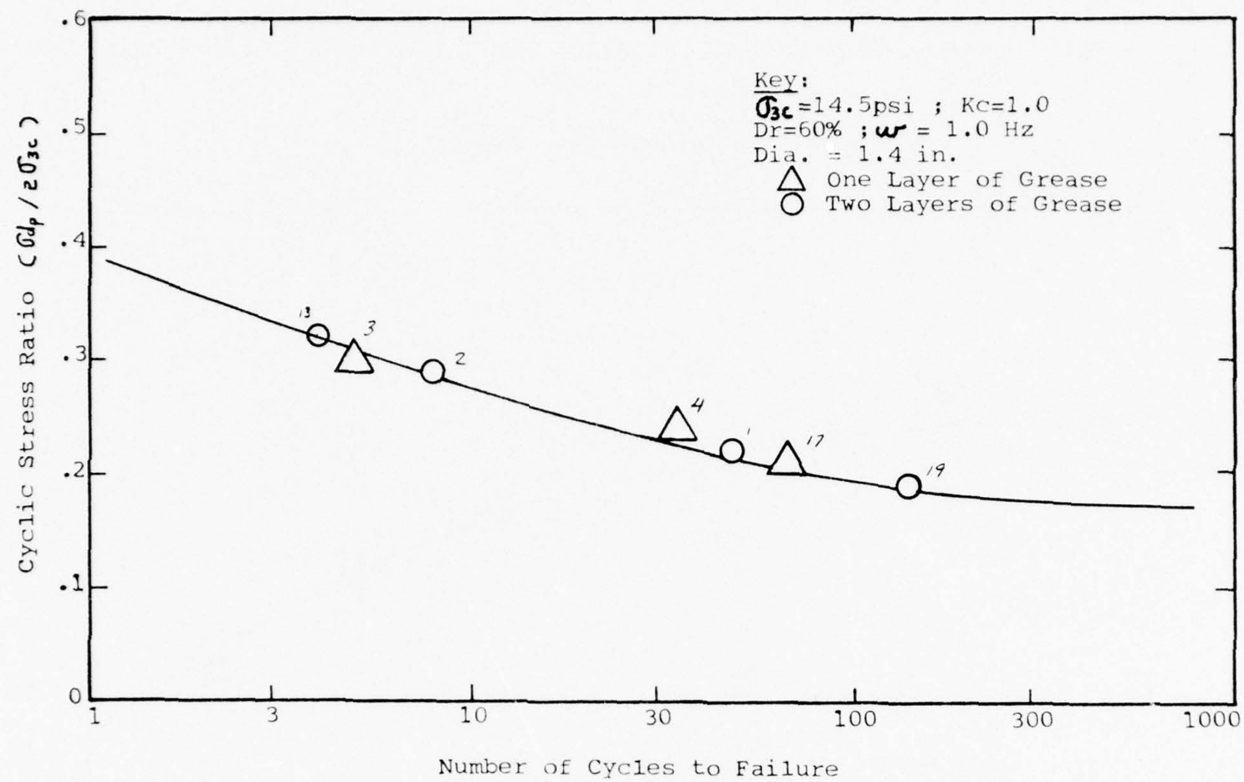


FIG. 2-9 COMPARISON OF STRENGTH DATA FROM CYCLIC TRIAXIAL TESTS ON LOOSE MONTEREY SAND- FRICTIONLESS ENDS, ONE AND TWO LAYERS OF GREASE

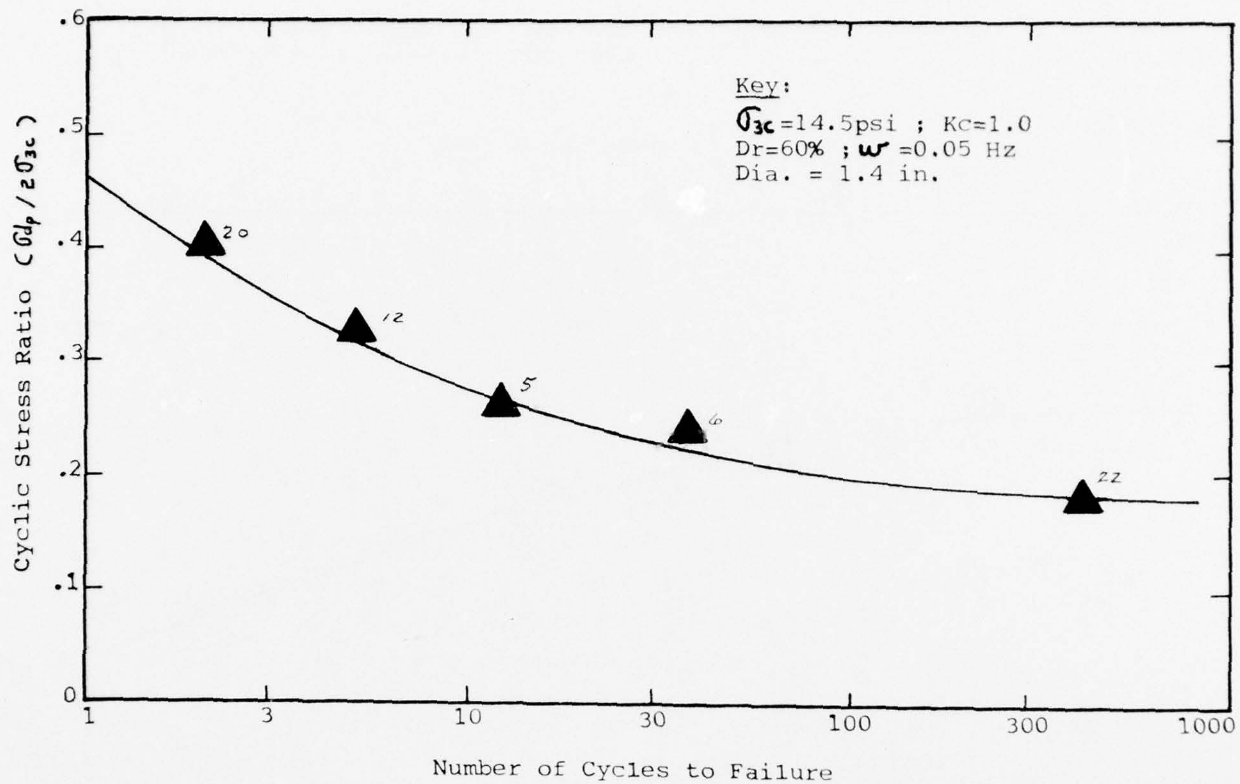


FIG. 2-10 STRENGTH DATA FROM CYCLIC TRIAXIAL TESTS ON LOOSE MONTEREY SAND - FRICTIONLESS ENDS

1.0 Hz, shows that there is no difference in cyclic strength as shown on Figure 2-11. This same comparison was made to determine the effect of frequency on regular ends. The strength data shown in Figure 2-12 refers to regular end tests conducted at 0.05 Hz. Data from the two types of tests are compared on Figure 2-13. The results show no difference in the cyclic strength when tested at 1.0 Hz and at 0.05 Hz. Thus, changing the frequency at which a test is run does not effect the cyclic strength of loose Monterey Sand. Figure 2-14 shows that for tests run at 0.05 Hz, the same 15 percent increase in strength was gained by using frictionless ends as was gained in tests run at 1.0 Hz.

Effect of Sample Size on the Strength of Loose Sand

All test results presented thus far were for samples of 1.4 in. diam. and 3.4 in. high. In order to determine the effect of sample size on the cyclic strength of loose sand, a series of additional tests were conducted on samples with 2.8 in. diam. and 6.1 in. high. The larger samples were prepared in the same manner and tested under the same conditions of density and confining pressure as previously described for the smaller samples. Although it already had been shown that frequency had no effect on the cyclic strength, all 2.8 in. samples were run at the slow cyclic frequency of 0.05 Hz. This frequency was slower than a typical earthquake frequency, but was used to dispell any argumens that the test frequency may have been too large to allow good lubrication at the viscous grease ends.

A typical record of a cyclic triaxial test using frictionless ends on a 2.8 in. sample of loose sand is shown on Figure 2-15. In this

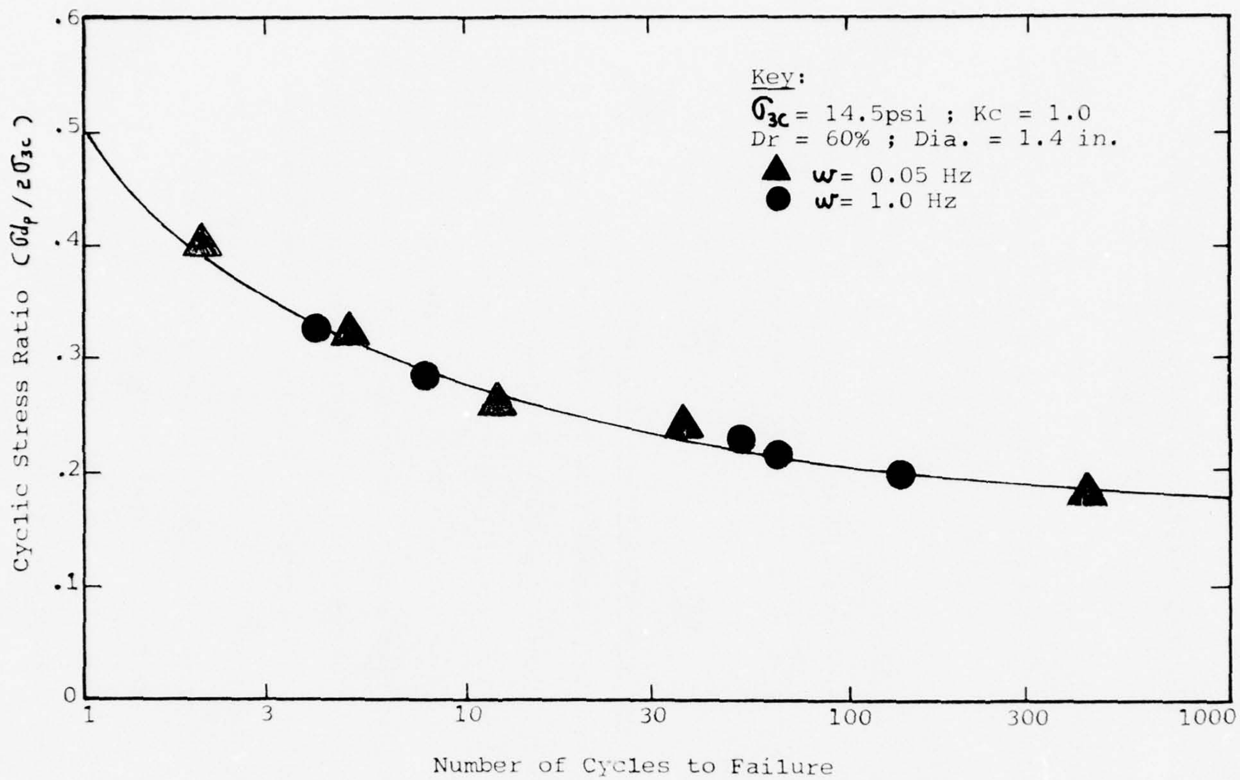


FIG. 2-11 EFFECT OF FREQUENCY ON THE CYCLIC TRIAXIAL STRENGTH OF LOOSE MONTEREY SAND - FRICTIONLESS ENDS

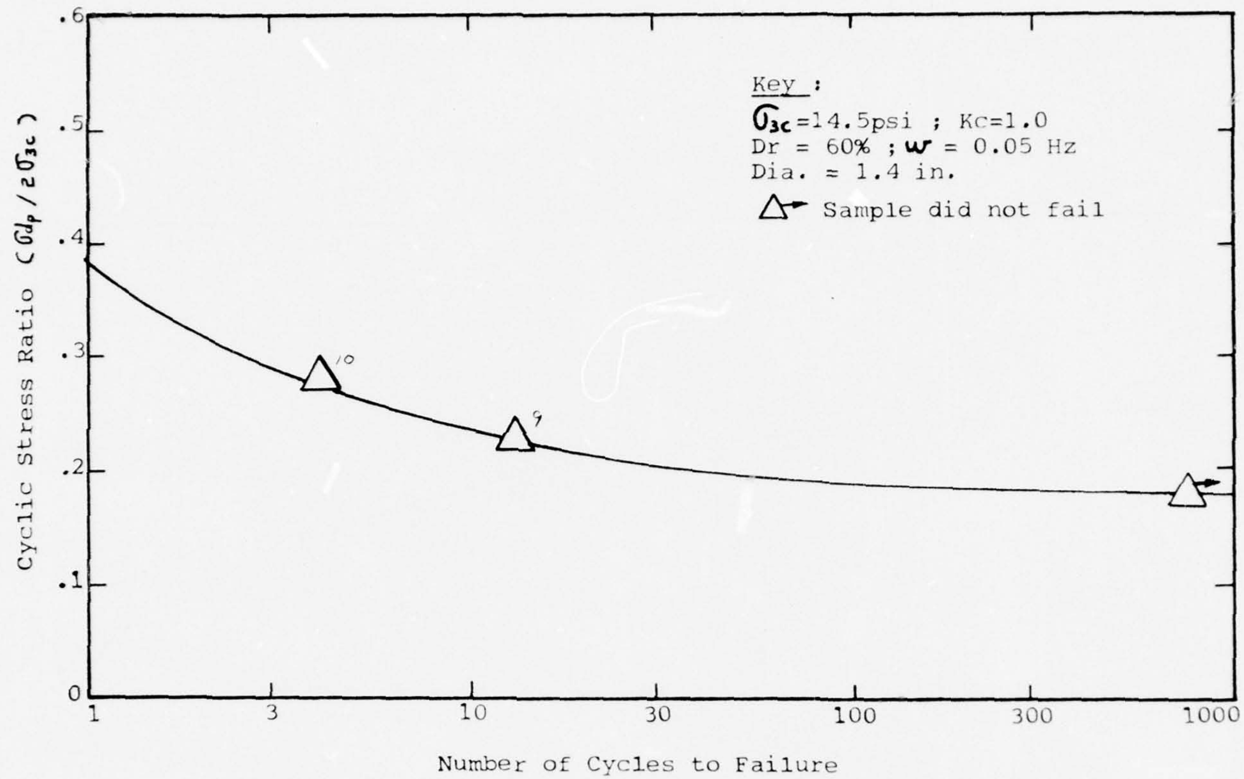


FIG. 2-12 STRENGTH DATA FROM CYCLIC TRIAXIAL TESTS ON LOOSE MONTEREY SAND - REGULAR ENDS

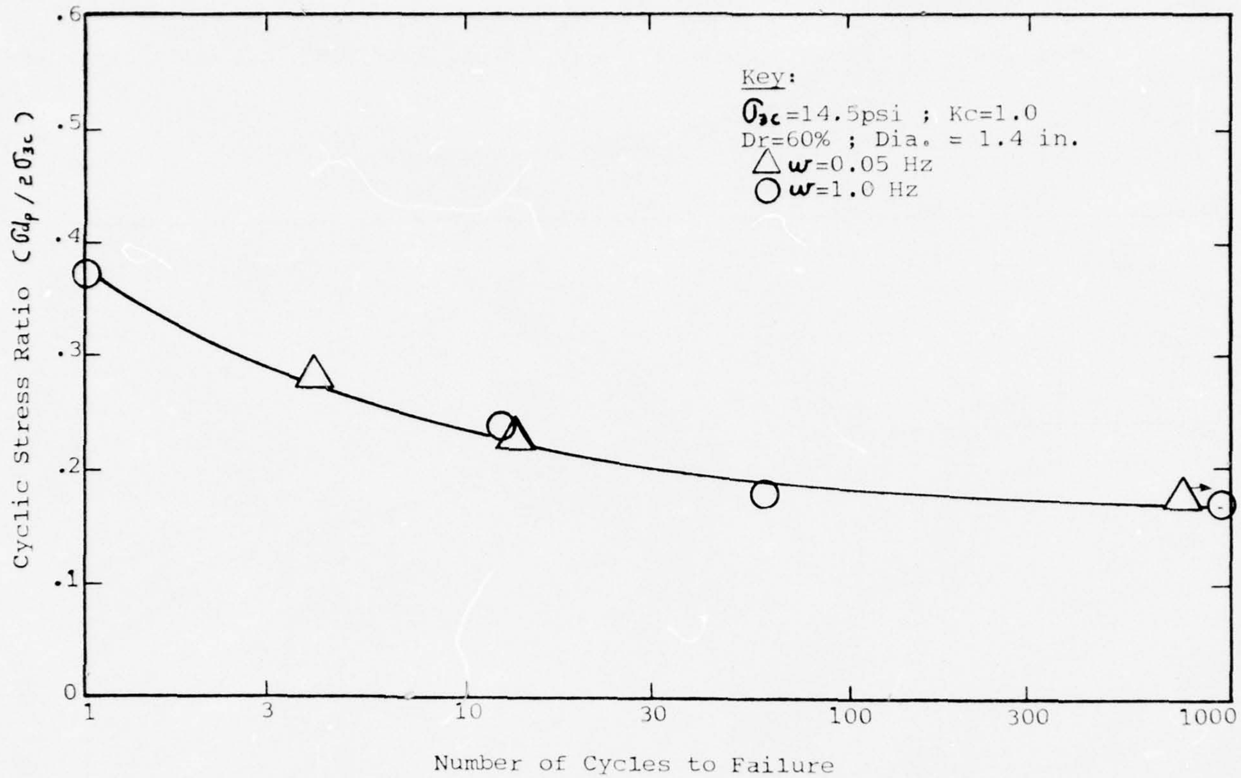


FIG. 2-13 EFFECT OF FREQUENCY ON THE CYCLIC TRIAXIAL STRENGTH OF LOOSE MONTEREY SAND - REGULAR ENDS

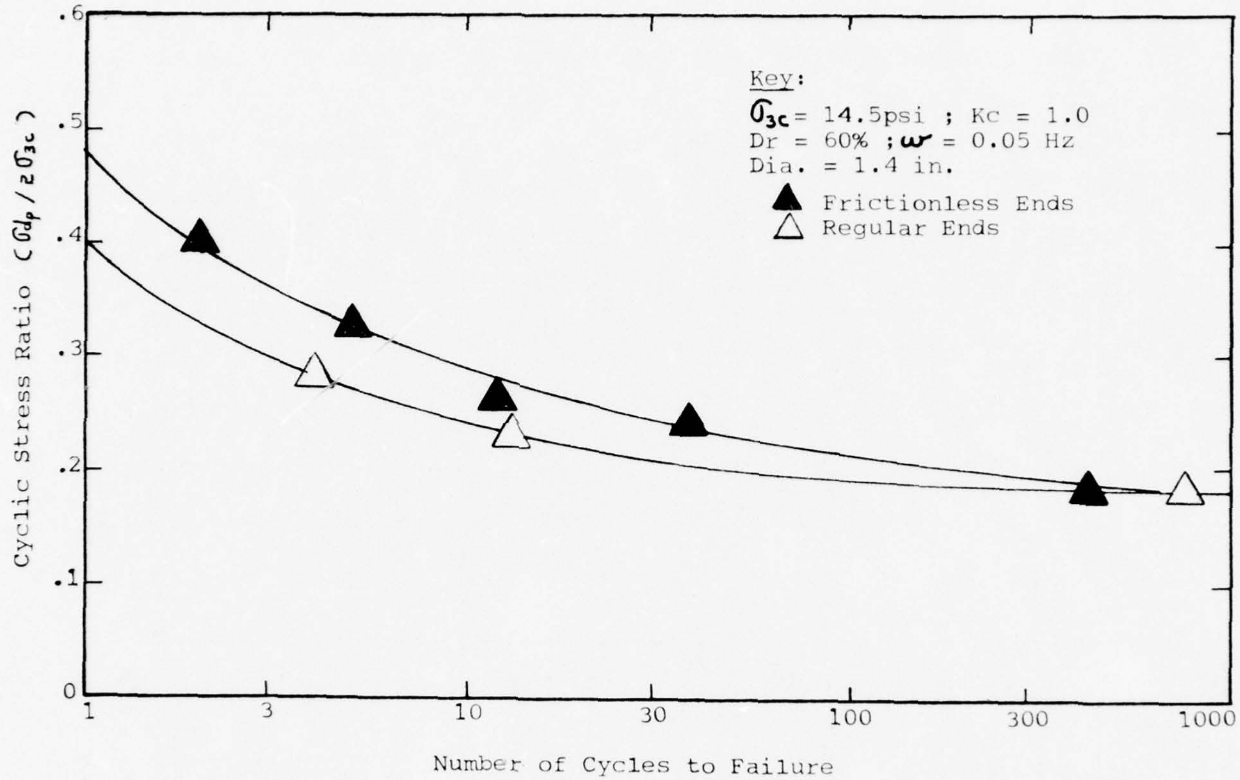


FIG. 2-14 COMPARISON OF STRENGTH DATA FROM CYCLIC TRIAXIAL TESTS ON LOOSE MONTEREY SAND

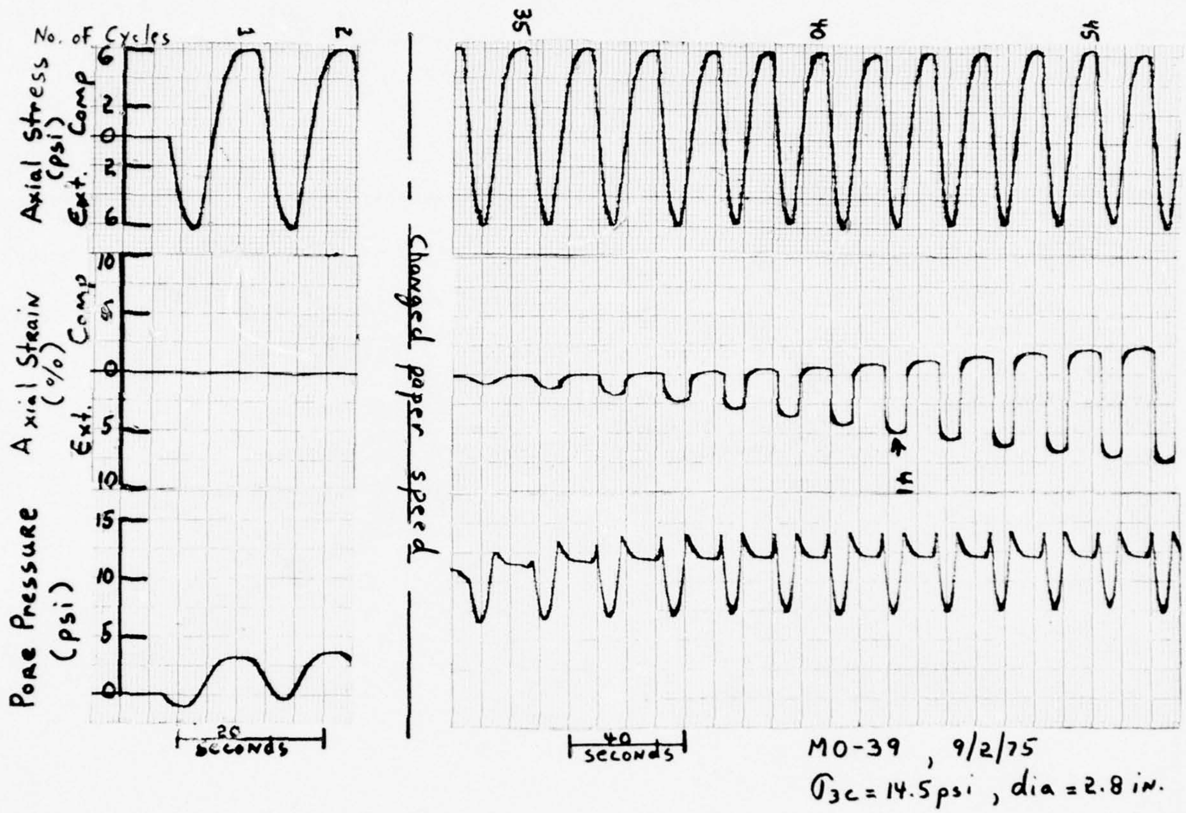


FIG. 2-15 RECORD OF A TYPICAL CYCLIC TRIAXIAL TEST ON LOOSE MONTEREY SAND, FRICTIONLESS ENDS, 2.8 INCH SAMPLE

test, a constant amplitude cyclic deviator stress, σ_{dp} = 6.1 psi was applied to the sample of saturated sand under a confining pressure of 14.5 psi. The resulting changes in axial strain and pore pressure were recorded as shown. It should be noted that this record is very similar to records shown previously for 1.4 in. samples. Again, the sample did not strain until the pore pressure reached a point close to the confining pressure. In addition, once the sample did begin to strain, it reached the 5 percent double amplitude failure strain within the next 10 cycles. In this case, failure occurred in 41 cycles.

Strength data for a number of 2.8 in. diam. samples with frictionless ends are shown in Figure 2-16. A comparison of the frictionless end data from 2.8 in. diam. and 1.4 in. diam. samples is shown on Figure 2-17. There is little difference in cyclic strength of these two sizes of samples when tested with frictionless ends. Additional tests were conducted on 2.8 in. samples with regular ends. The results are presented on Figure 2-18 and the results compared with 1.4 in. diam. samples are on Figure 2-19. Again, there was no observed significant difference in cyclic strength of regular end samples of 1.4 and 2.8 in. diam.

Effect of Varying the K_c Ratio on the Strength of Loose Sand

In order to show the effect of varying anisotropic consolidation stress ratio K_c on the cyclic strength of loose sand, three series of tests were performed using K_c = 1.0, 1.5 and 2.0. Each series of tests were performed for both frictionless and regular ends on 2.8 in. diam. samples. The samples were prepared in the same manner and run at the same density and confining pressure as previously described.

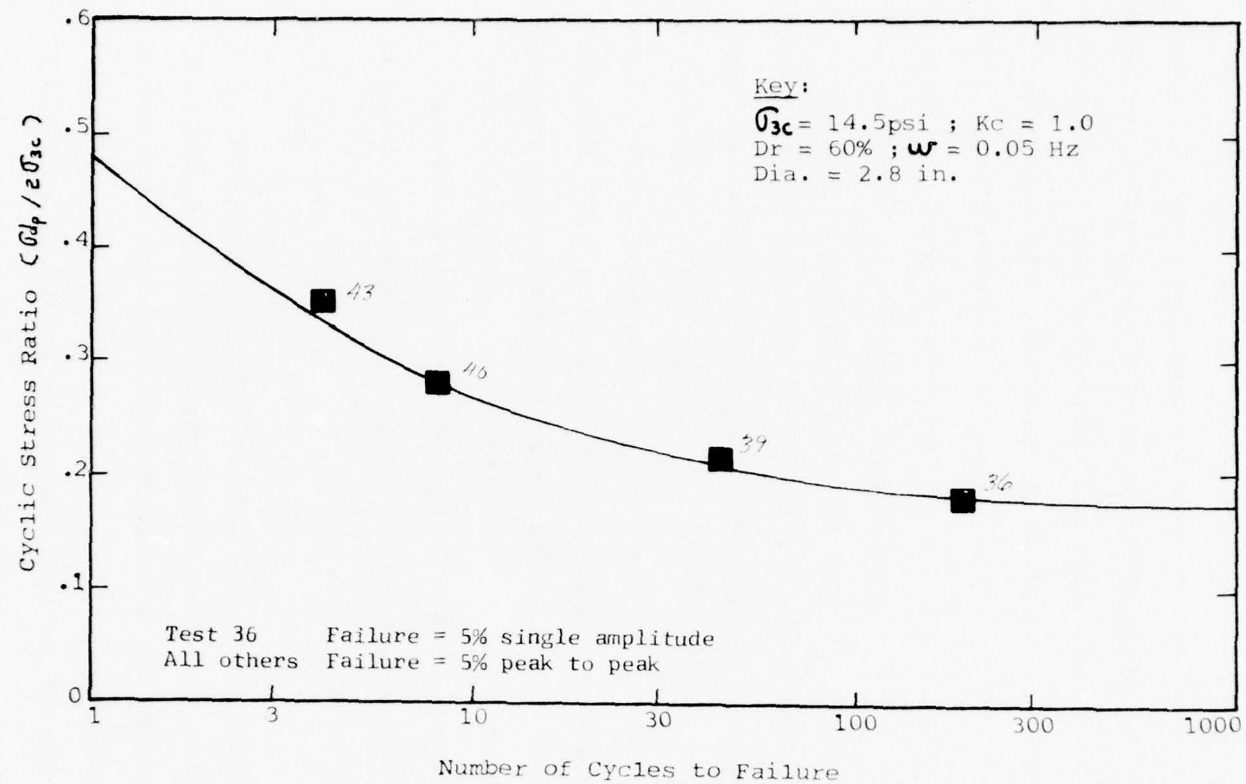


FIG. 2-16 STRENGTH DATA FROM CYCLIC TRIAXIAL TESTS ON LOOSE MONTEREY SAND - FRICTIONLESS ENDS

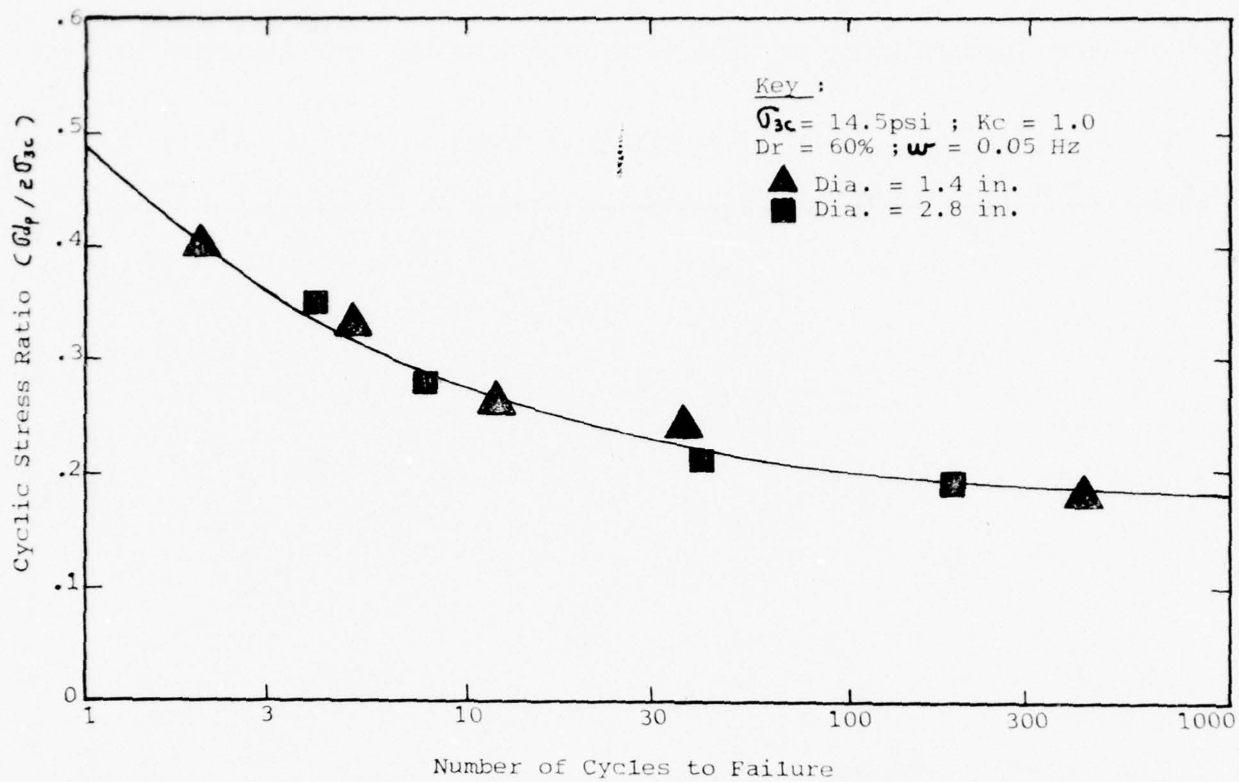


FIG. 2-17 EFFECT OF SAMPLE SIZE ON THE CYCLIC TRIAXIAL STRENGTH OF LOOSE MONTEREY SAND - FRICTIONLESS ENDS

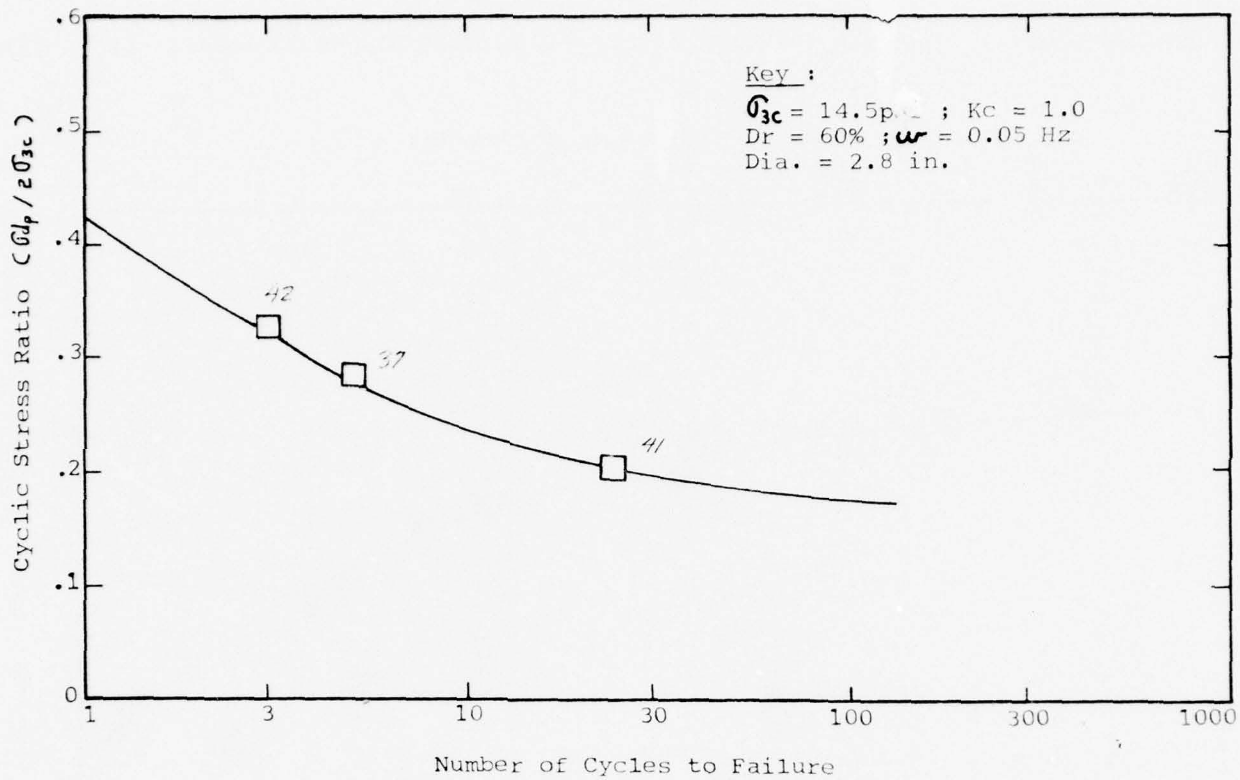


FIG. 2-18 STRENGTH DATA FROM CYCLIC TRIAXIAL TESTS ON LOOSE MONTEREY SAND - REGULAR ENDS

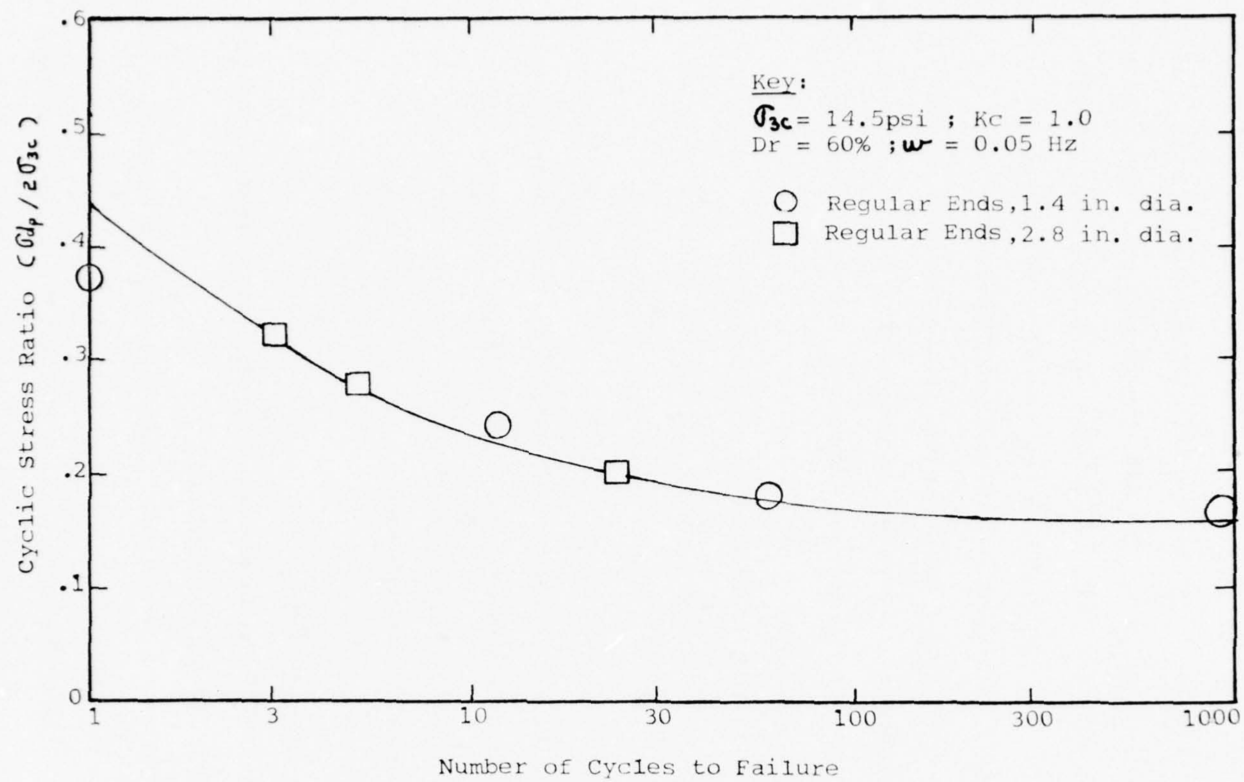


FIG. 2-19 COMPARISON OF STRENGTH DATA FROM CYCLIC TRIAXIAL TESTS ON LOOSE MONTEREY SAND

The results of cyclic loading tests on samples with both friction and regular ends for Kc = 1.0 have been presented on Figures 2-17 through 2-19 respectively. The data showed that the samples with frictionless ends were 10 to 20 percent stronger than those with regular ends. The second series of tests were run with K_c = 1.5. Strength data for this series are shown on Figures 2-20 and 2-21 for tests on samples with frictionless and regular ends respectively. A comparison of these two curves is presented on Figure 2-22 and shows that tests with frictionless ends were about 15 percent stronger than tests with regular ends.

Data for tests performed at Kc = 2.0 are shown in Figures 2-23 through 2-25. In this case, tests run with frictionless ends were 20-30 percent stronger than tests run with regular ends.

Previous studies by Lee and Seed (3) have shown that increasing Kc increases the cyclic strength of saturated sands. The data from the studies described herein show that for clean saturated sands, the higher the Kc ratio the greater the cyclic strength for frictionless ends as compared to regular ends.

Conclusions for Loose Sand

The results of the data shown thus far, for loose Monterey sand, leads to the following conclusions:

(1) The use of prongs in frictionless caps and bases does not effect the cyclic strength of loose Montery sand.

(2) For loose Monterey sand, the effect of frictionless ends lead to an increase in cyclic triaxial strength of about 10 to 20 percent as compared to tests with regular ends. Expressed another way,

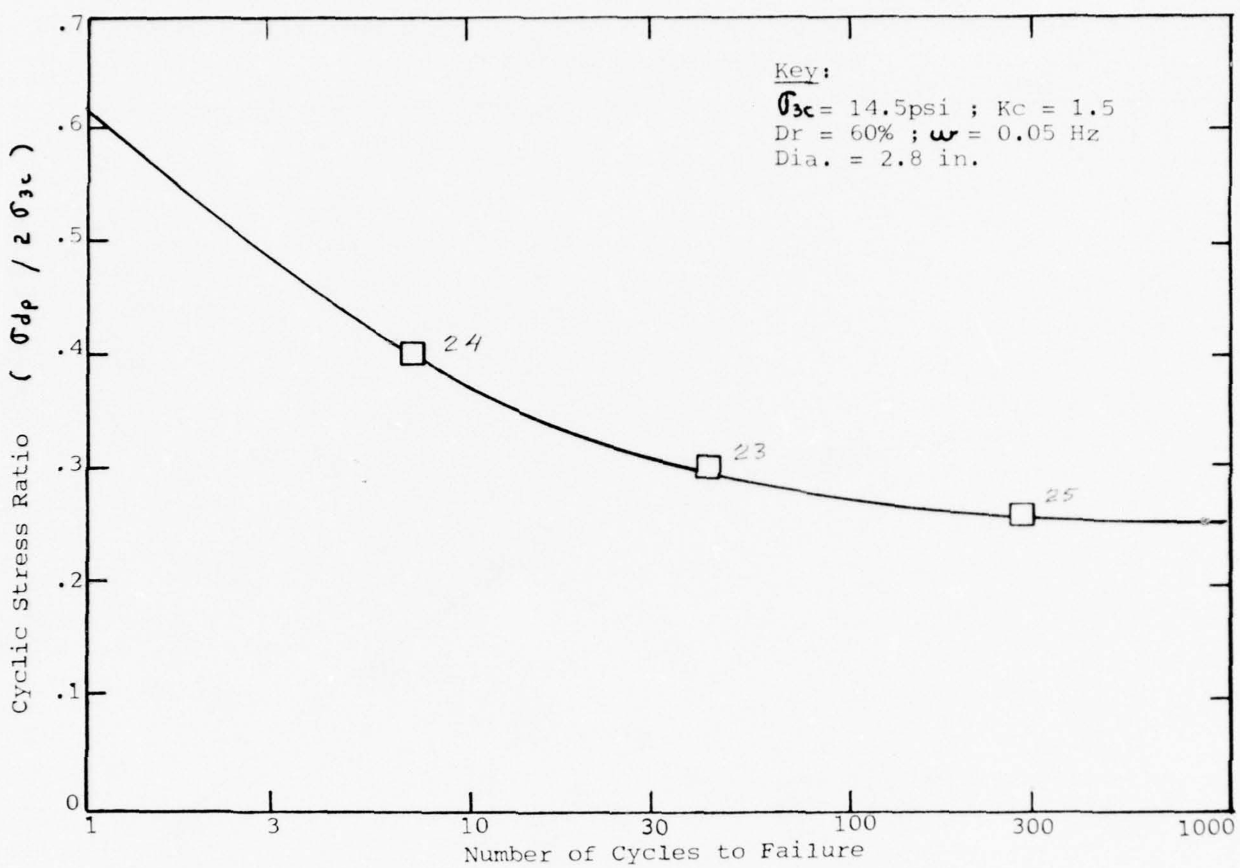


FIG. 2-20 STRENGTH DATA FROM TESTS ON LOOSE MONTEREY SAND - REGULAR ENDS, Kc = 1.5

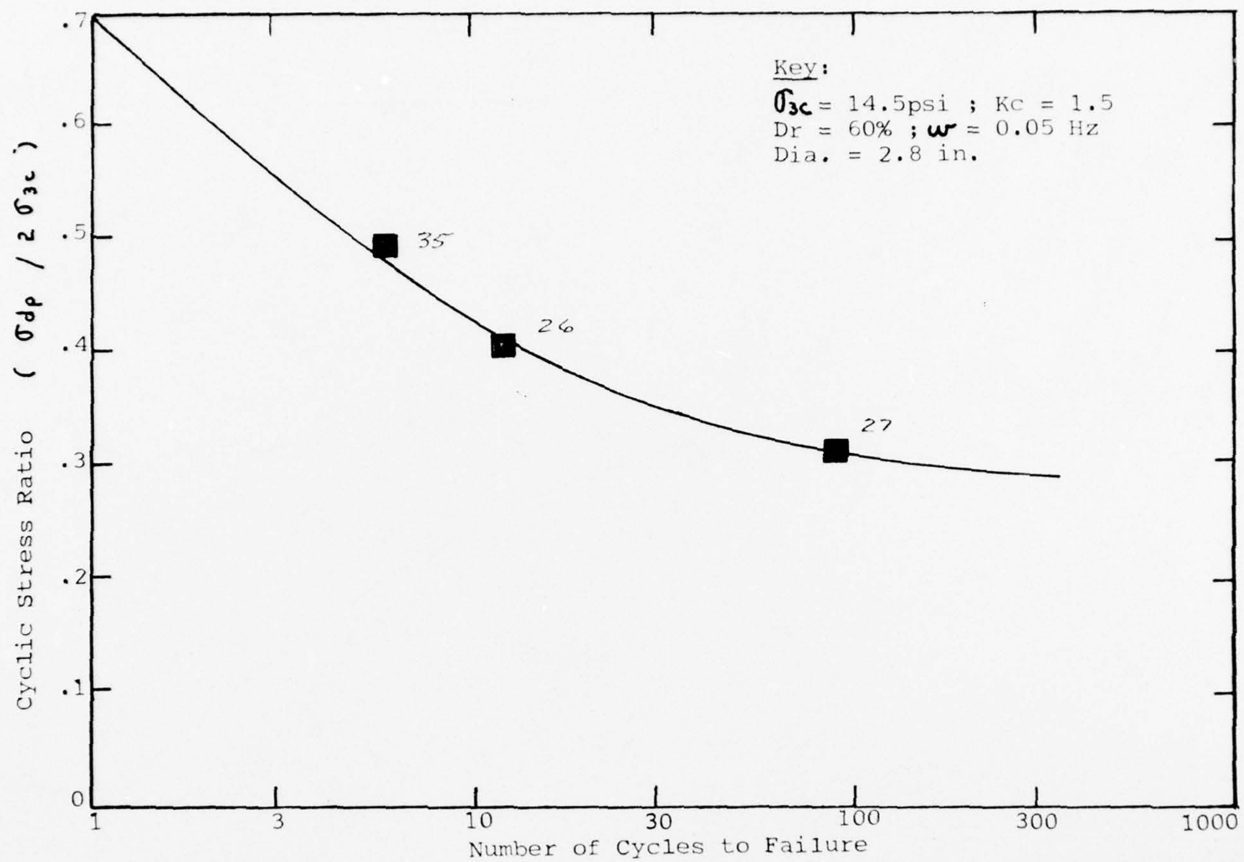


FIG. 2-21 STRENGTH DATA FROM TESTS ON LOOSE MONTEREY SAND-FRICTIONLESS ENDS, Kc = 1.5

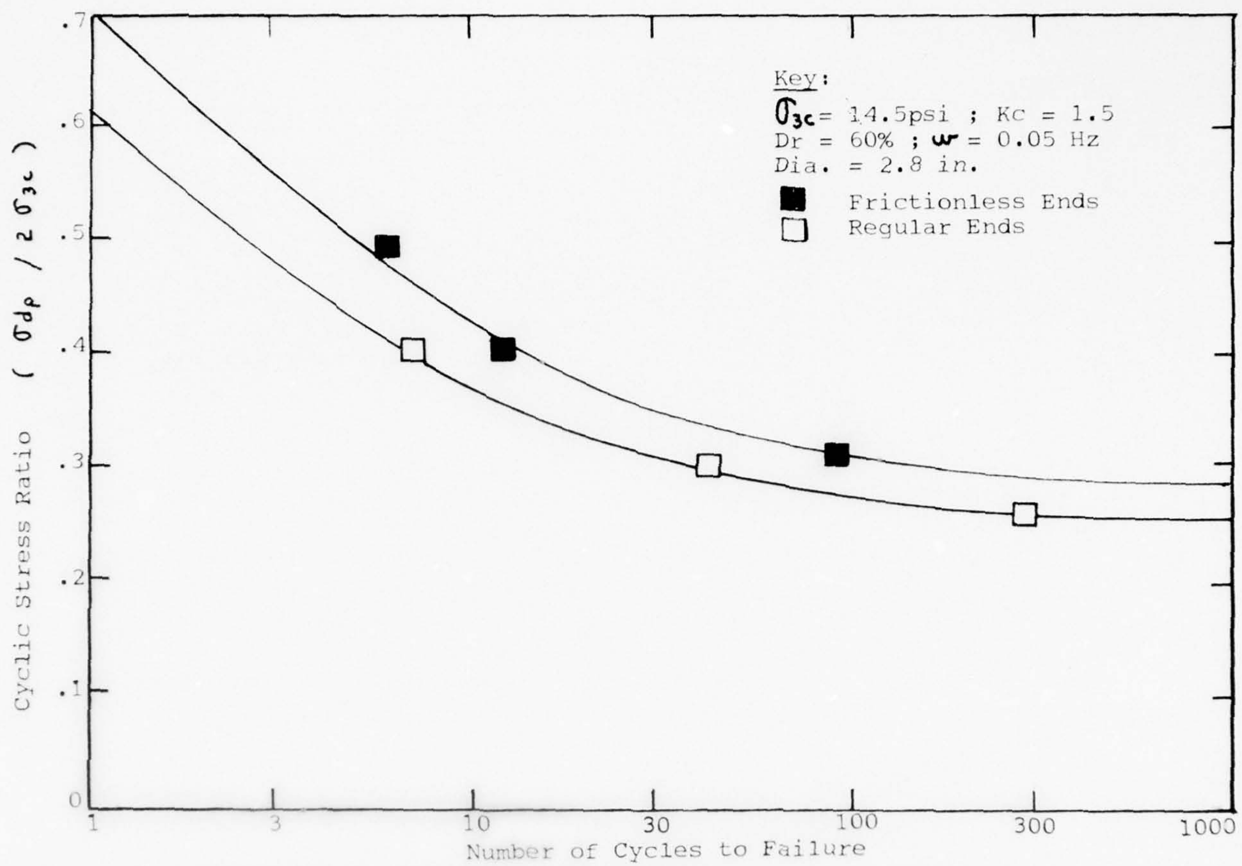


FIG. 2-22 COMPARISON OF STRENGTH DATA FROM TESTS ON LOOSE MONTEREY SAND , Kc =1.5

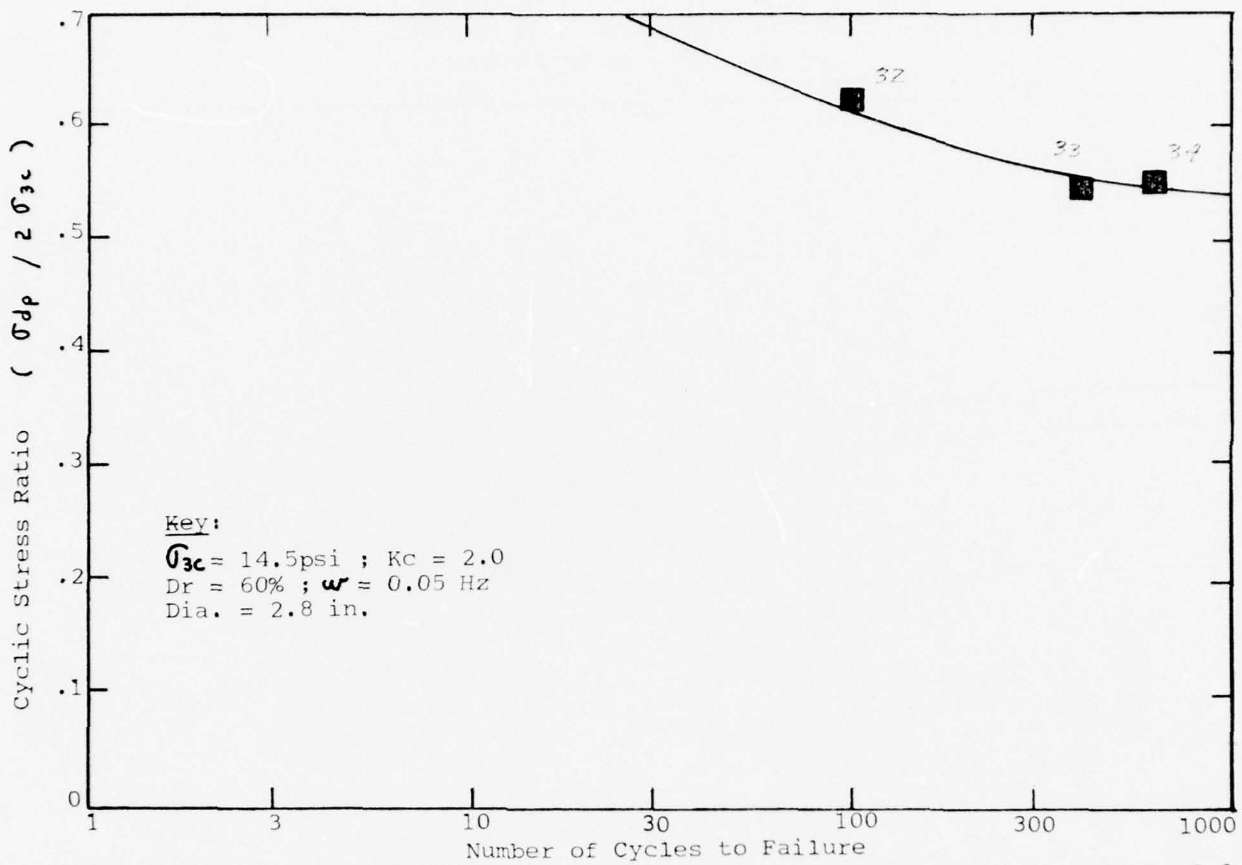


FIG. 2-23 STRENGTH DATA FROM TESTS ON LOOSE MONTEREY SAND - FRICTIONLESS ENDS, Kc=2.0

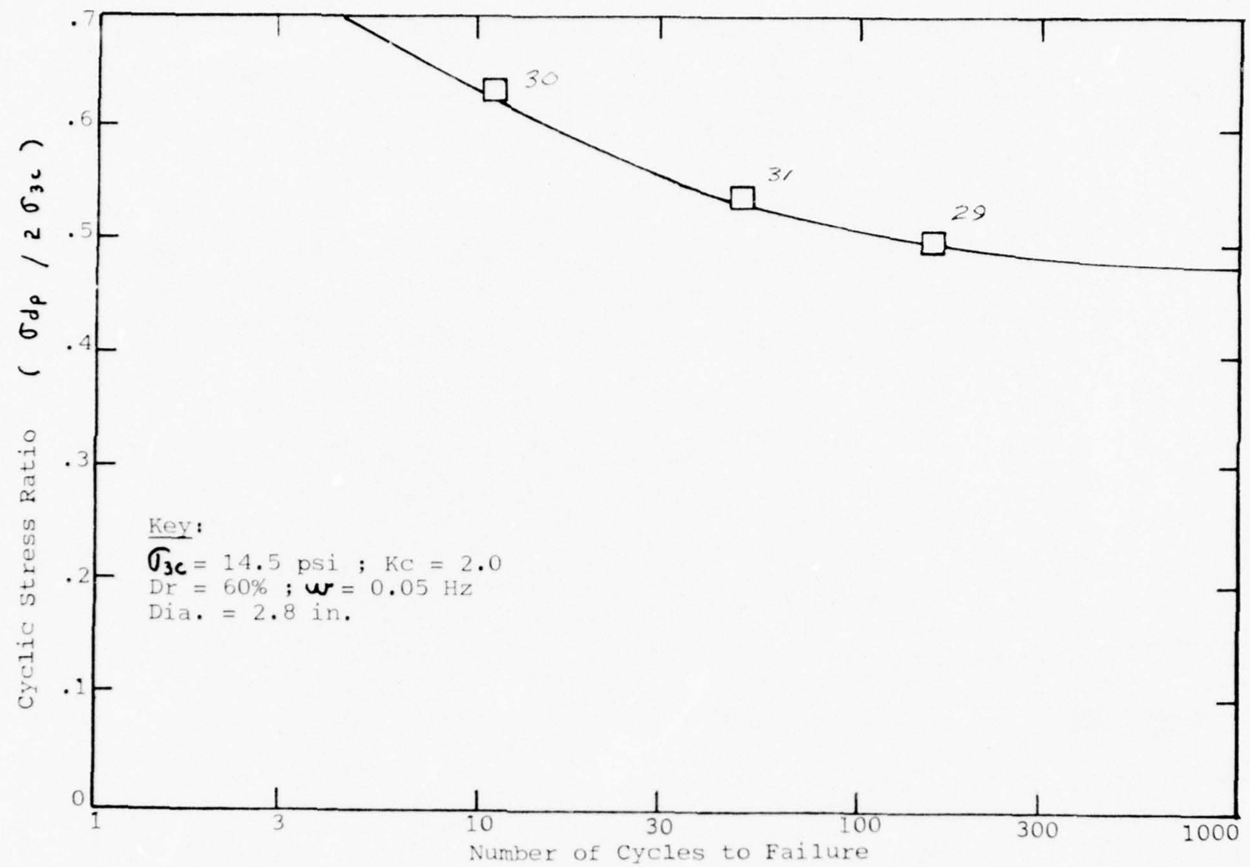


FIG. 2-24 STRENGTH DATA FROM TESTS ON LOOSE MONTEREY SAND - REGULAR ENDS, Kc = 2.0

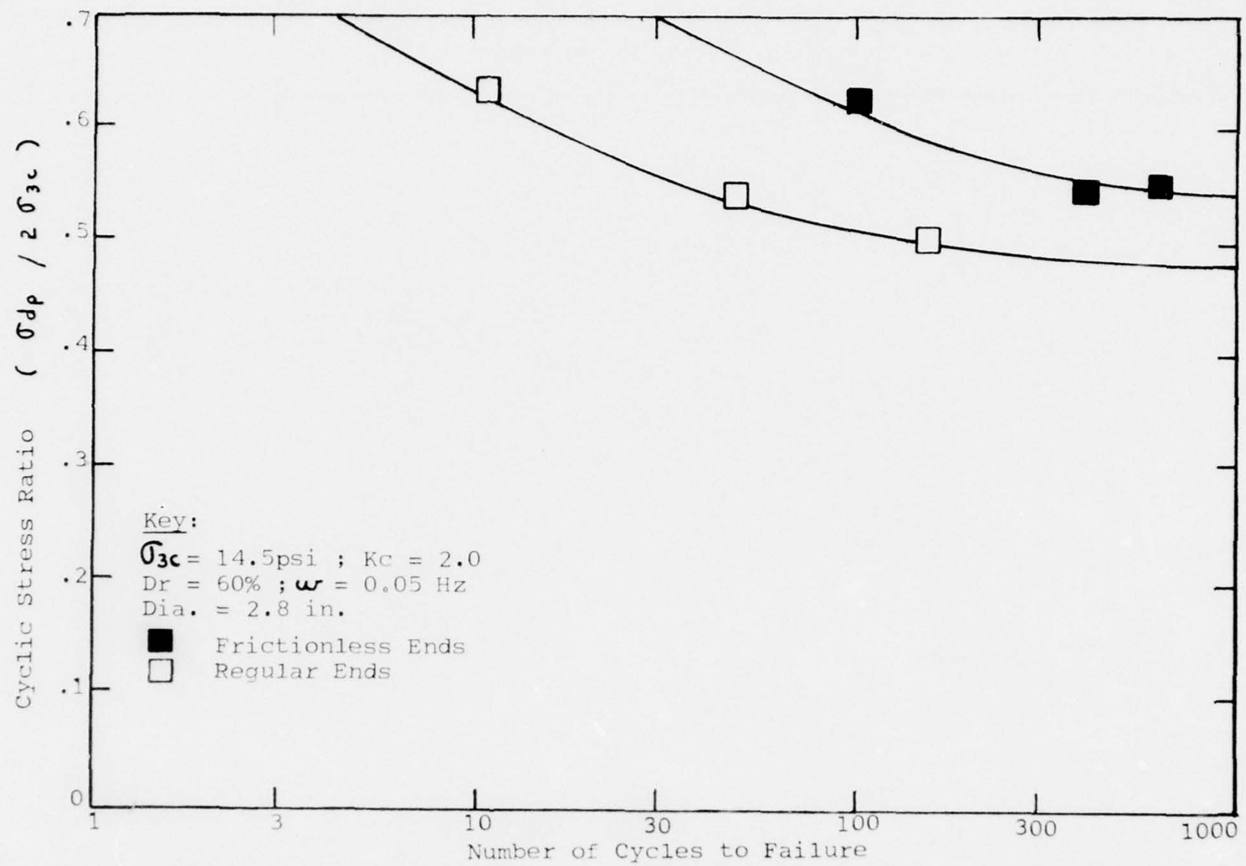


FIG. 2-25 COMPARISON OF STRENGTH DATA FROM TESTS ON LOOSE MONTEREY SAND , Kc = 2.0

at the same cyclic stress ratio samples with frictionless ends required about 1 order of magnitude more cycles than regular ends to cause initial liquefaction or failure at 5 percent double amplitude strain.

(3) Changing the frequency at which a test is run does not effect the cyclic strength of loose Monterey sand nor does it influence the effect of end restraints on the cyclic strength.

(4) There is little, if any, difference in the cyclic strength between tests run on this sand with 1.4 in. samples and tests performed on 2.8 in. samples.

(5) Increasing the Kc ratio increases both the cyclic strength and the effect of frictionless ends. By increasing Kc from 1.0 to 2.0, the effect of frictionless ends was to increase the cyclic strength from 15 to 30 percent over tests with regular ends.

A summary of all the results on loose Monterey sand is presented in Figures 2-26 and 2-27.

Behavior of Dense Sand

Two series of tests were performed on dense Monterey sand. The first series included tests on samples with frictionless and regular ends at a relative density of 90 percent. The results of these tests were inconclusive for reasons described in the following section. A second series of tests, using both frictionless and regular ends was then conducted at a relative density of 80 percent. Each test series was performed at a cyclic frequency of 0.05 Hz and using an effective confining pressure of 14.5 psi. The samples were prepared in the same manner as described for loose samples.

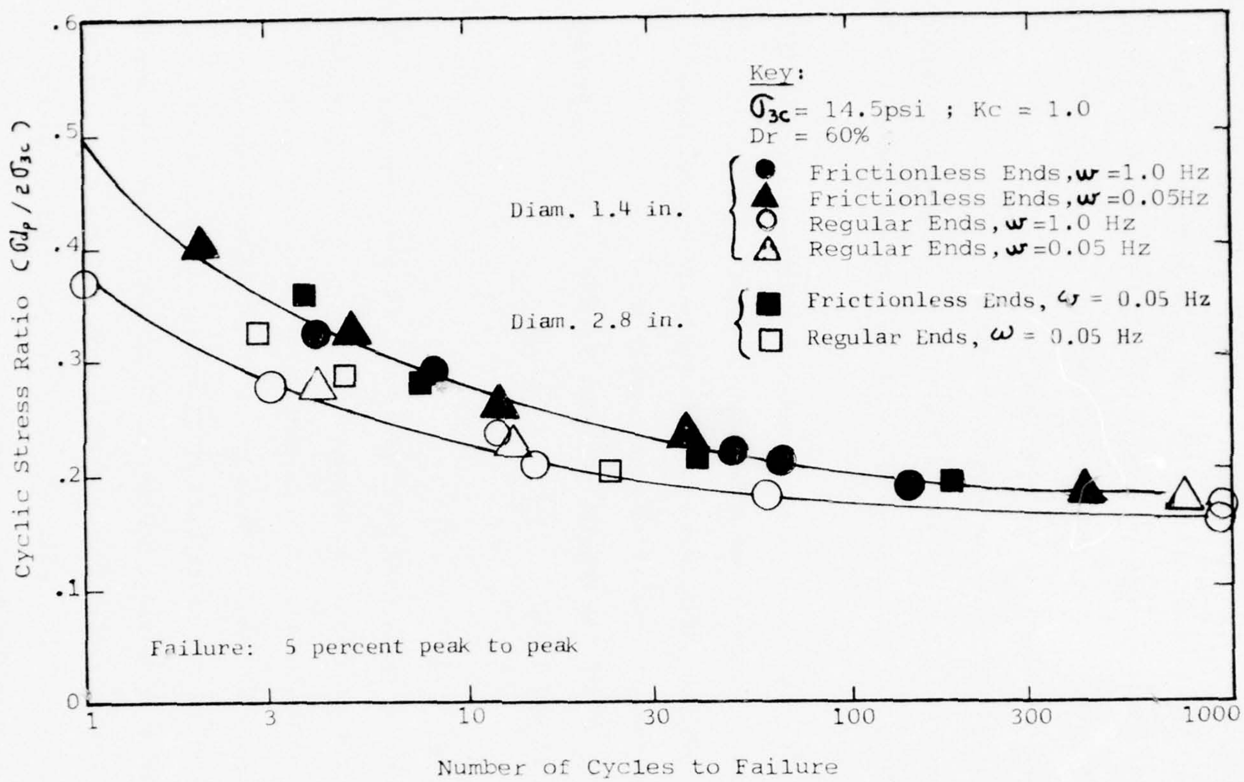


FIG. 2-26 SUMMARY OF STRENGTH DATA FROM CYCLIC TRIAXIAL TESTS ON LOOSE MONTEREY SAND , Kc = 1.0

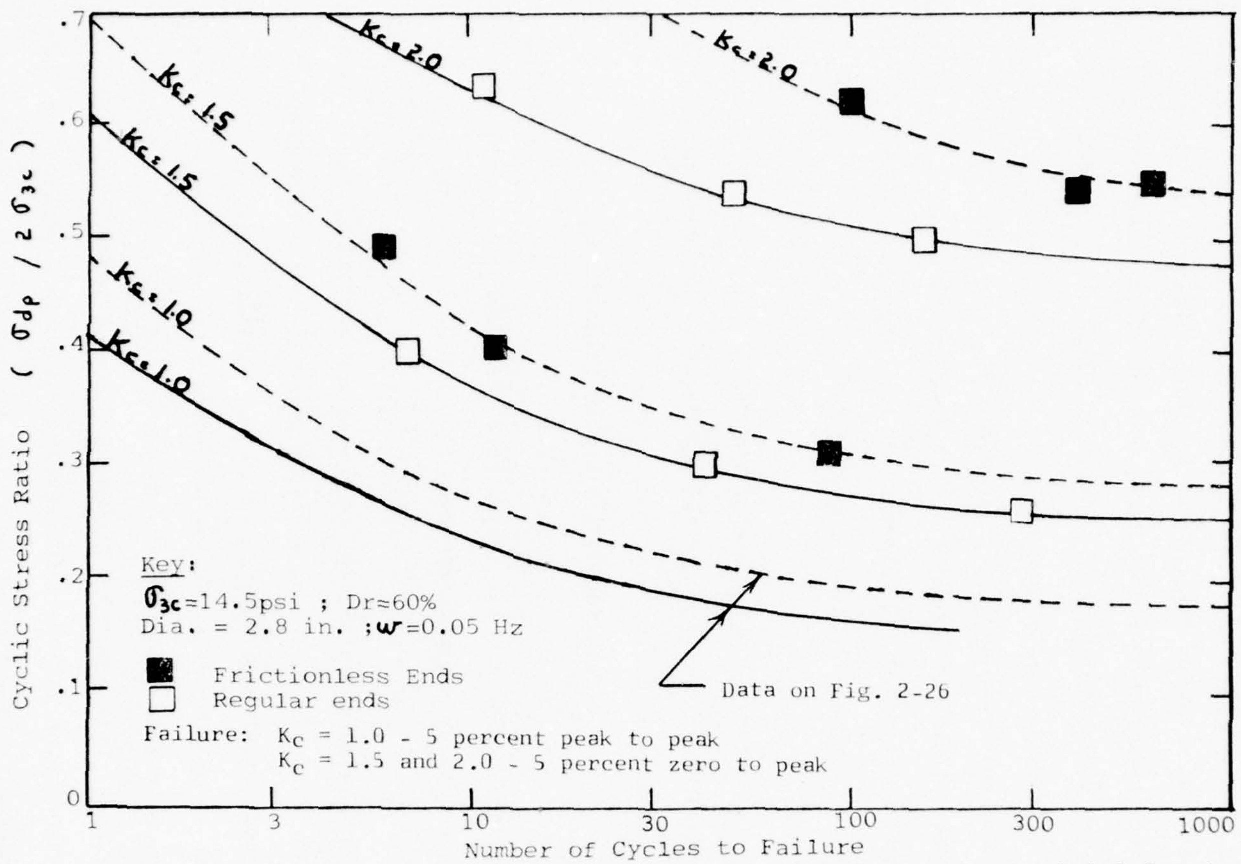


FIG. 2-27 SUMMARY OF STRENGTH DATA FROM TESTS ON LOOSE MONTEREY SAND, Kc=1.0,1.5,2.0

Effect of End Restraint on the Strength of Dense Sand (Dr = 90 Percent)

As stated above, the results for this series of tests were inconclusive, As shown on Figure 2-28, the data points for individual tests are so scattered that no definite conclusions could be drawn. In an attempt to unscramble the data, failure criteria of 1, 2 and 5 percent single and double amplitude strains were used. However, for each attempted failure criteria, the scatter and the lack of definitive conclusions remained. The reason for such inconclusive results was due to the high axial stress required to cause failure in a reasonable number of cycles.

It is interesting to note at this point that very high dilatancy tendencies were exhibited for all the samples tested in this series, with regular and frictionless ends. This may be seen on Figure 2-29, which shows a typical record of a cyclic triaxial test, using regular ends, on dense sand (Dr = 90 percent). In this test, a cyclic deviator stress, σ_{dp} = 25 psi, was used. Note the great dilatancy effect in that the pore pressure decreases well below zero. In fact, the pore pressure trace goes off the scale. Thus, decreasing the pore pressure by such a large amount during the extension portion of the cycle leads to higher transient effective stress and high cyclic strength. Thus, the sample strains at a very slow rate. Finally, when the sample started to approach the failure strain of 5 percent, the axial stress was so great that it pulled the sample off of its ends. As shown on Figure 2-29, this "necking" phenomena occurred in the 108th cycle. This sequence of failure deformation was also photographed and the photographs are shown on Figures 2-30 and 2-31 for both regular and frictionless ends respectively.

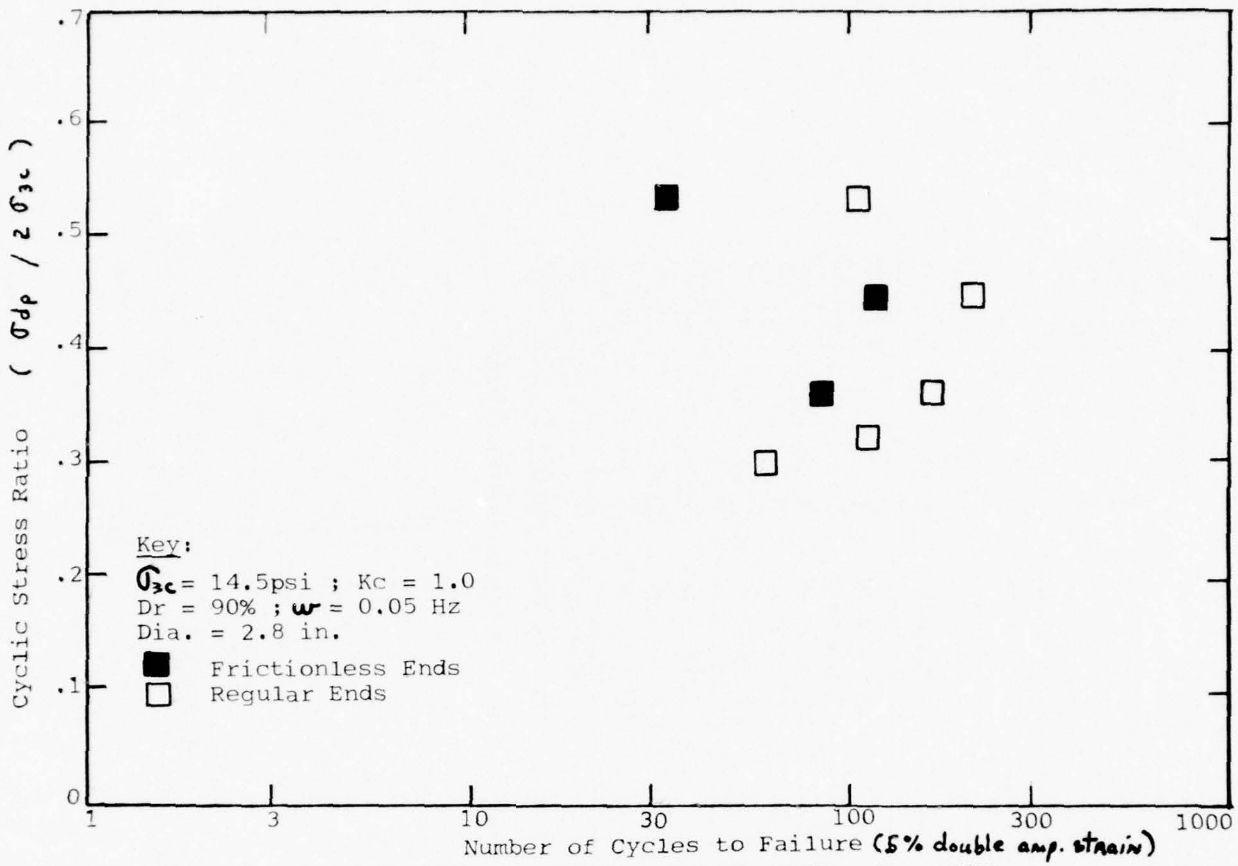


FIG. 2-28 STRENGTH DATA FROM TESTS ON DENSE MONTEREY SAND , Dr = 90%

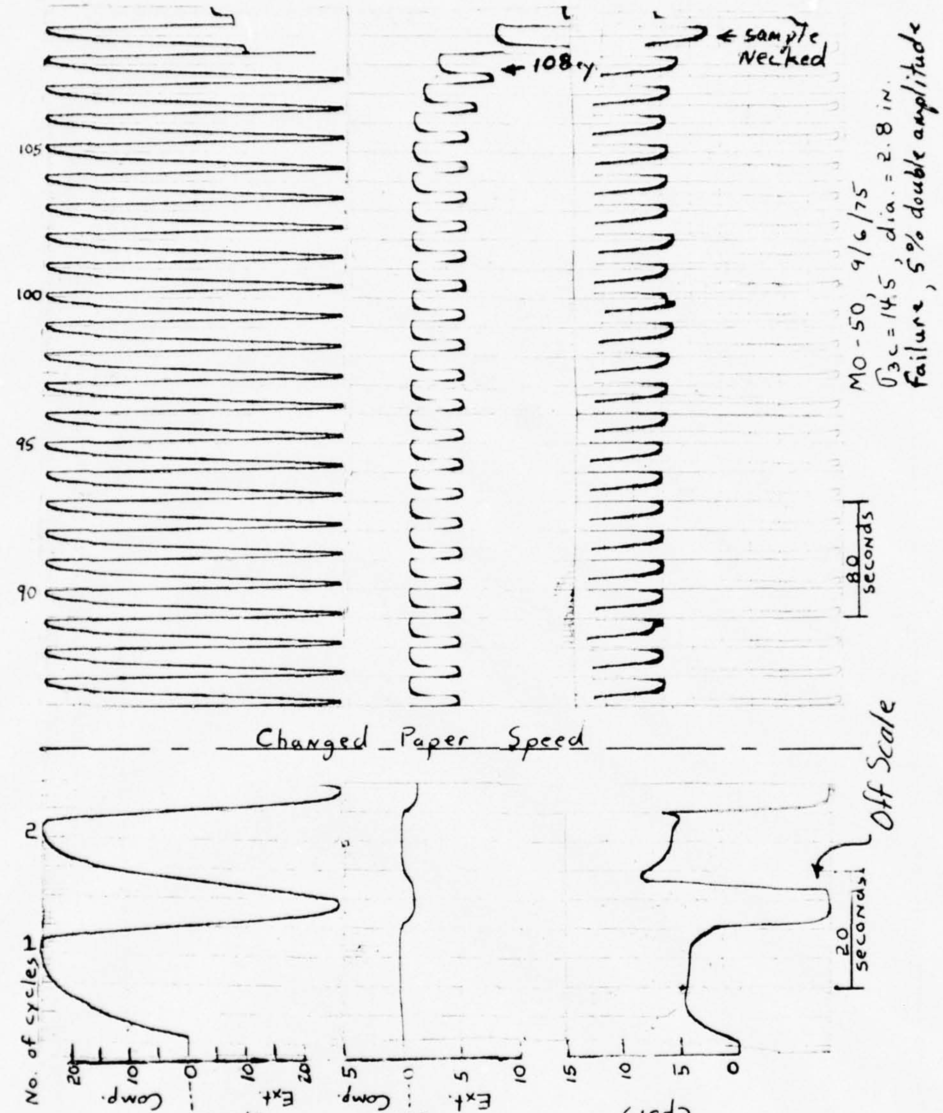
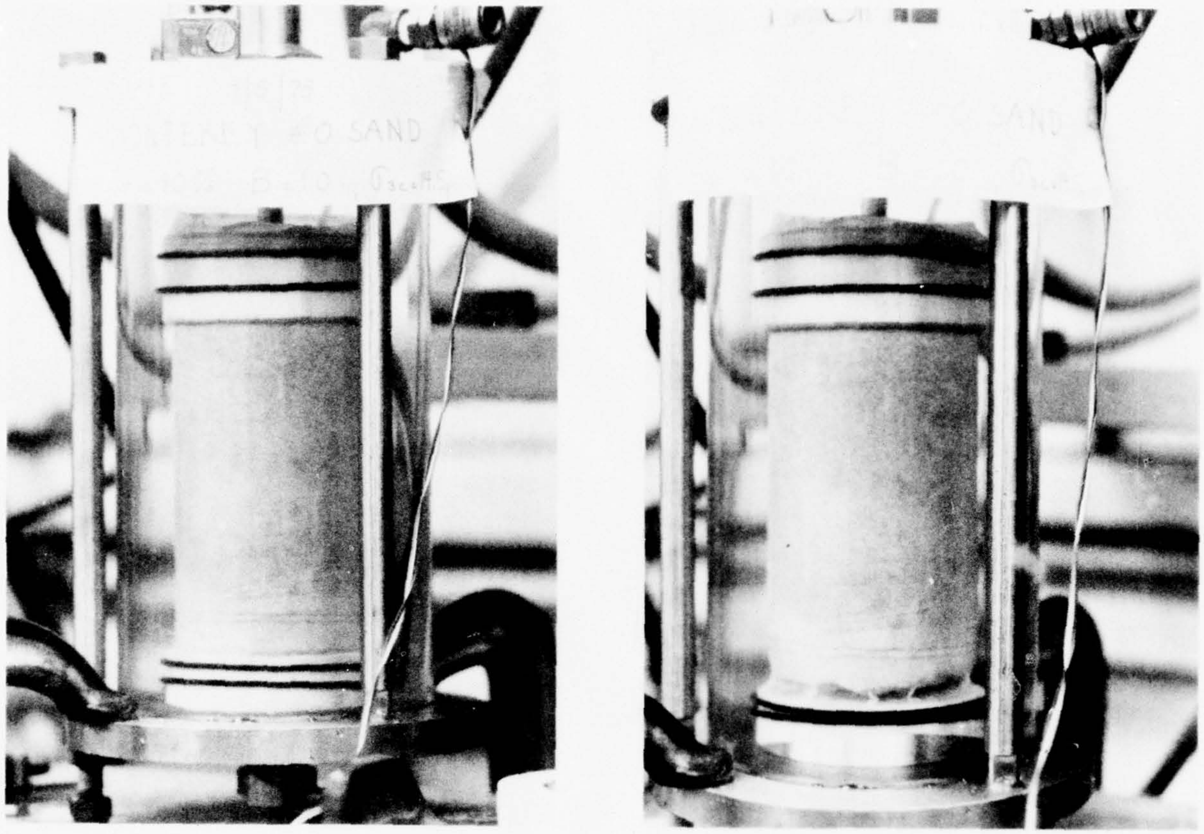


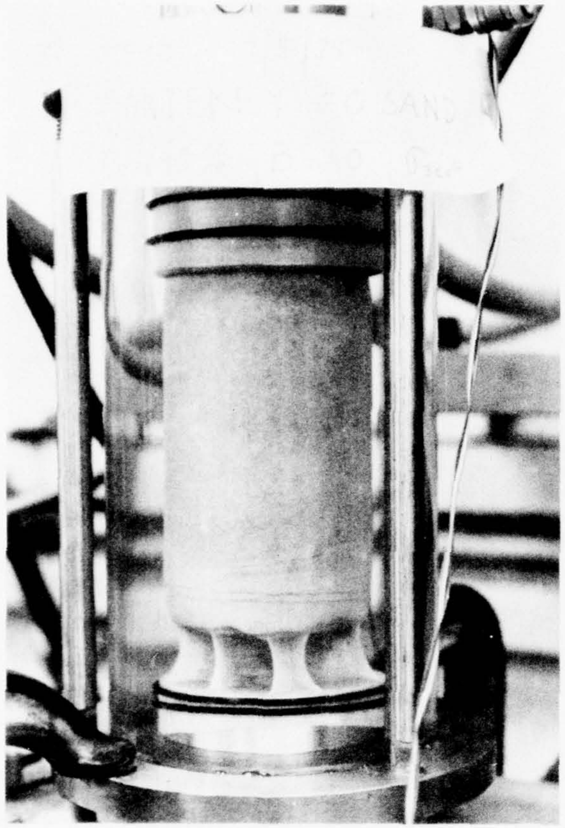
FIG.2-29 RECORD OF A TYPICAL CYCLIC TRIAXIAL TEST ON DENSE MONTEREY SAND, REGULAR ENDS



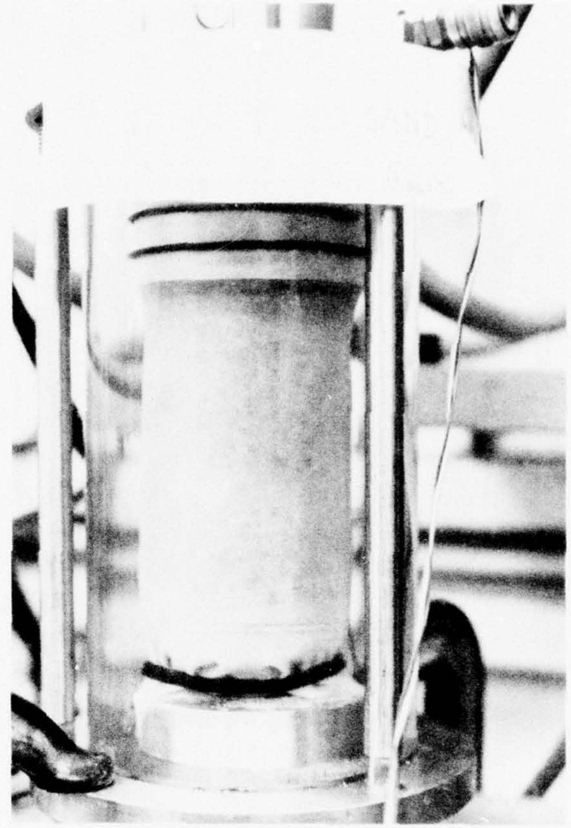
SAMPLE READY FOR TESTING

EXTENSION

FIG. 2-30 SEQUENCE OF FAILURE FOR A SAMPLE OF DENSE MONTEREY SAND - FRICTIONLESS ENDS. Dr = 90%

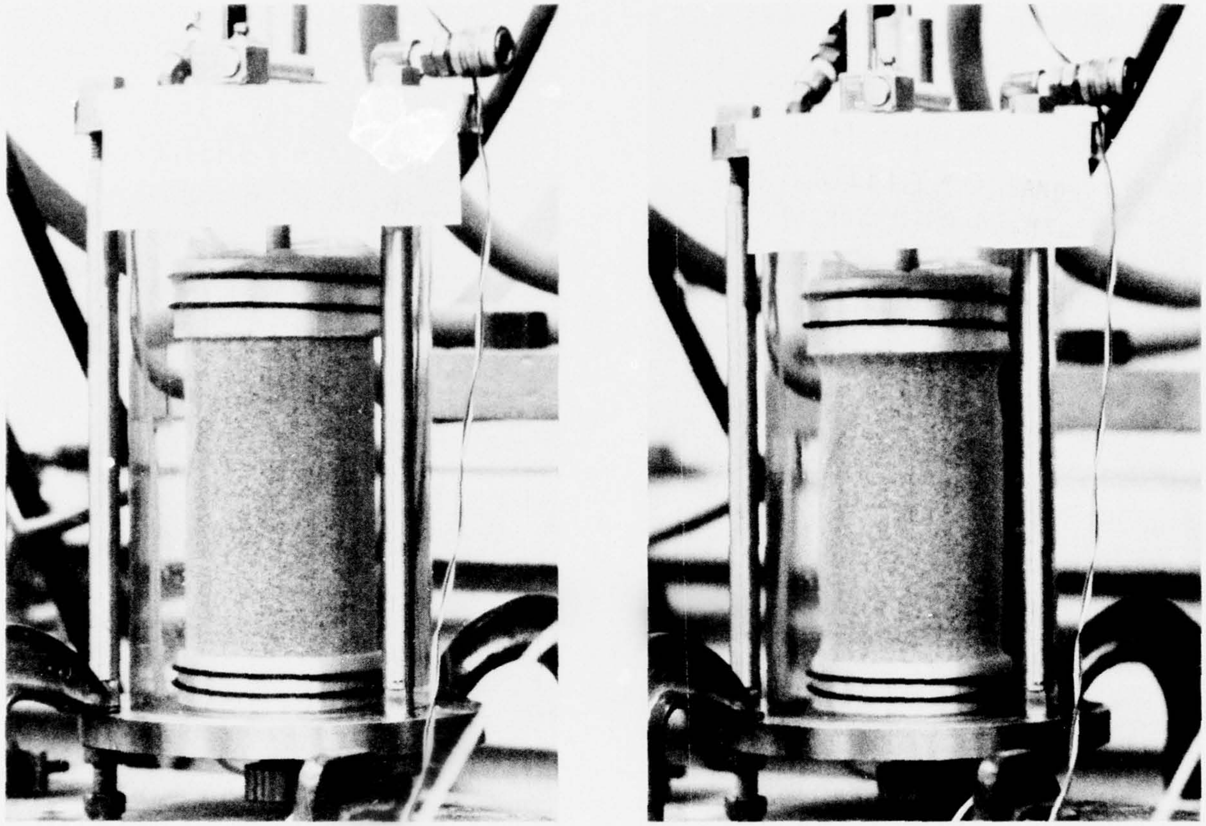


EXTENSION



EXTENSION

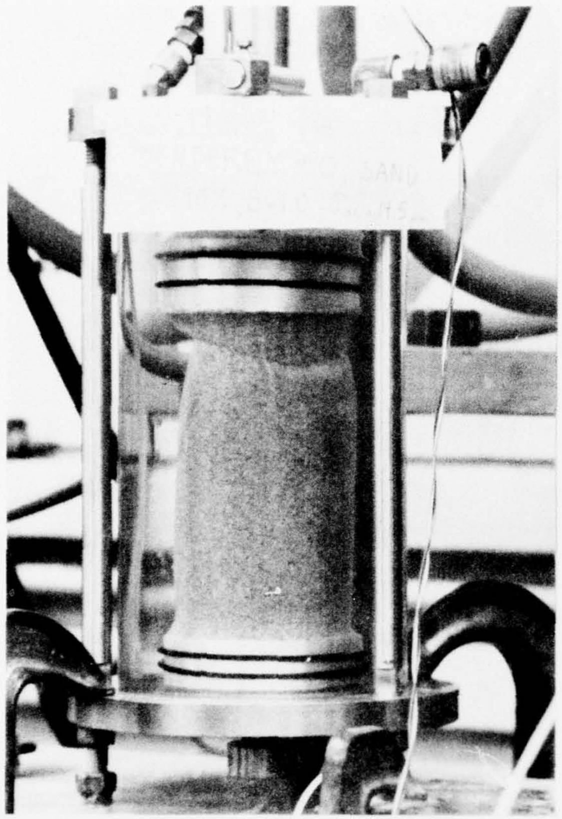
FIG. 2-30(Contd.) SEQUENCE OF FAILURE FOR A SAMPLE OF DENSE MONTEREY SAND - FRICTIONLESS ENDS. Dr = 90%



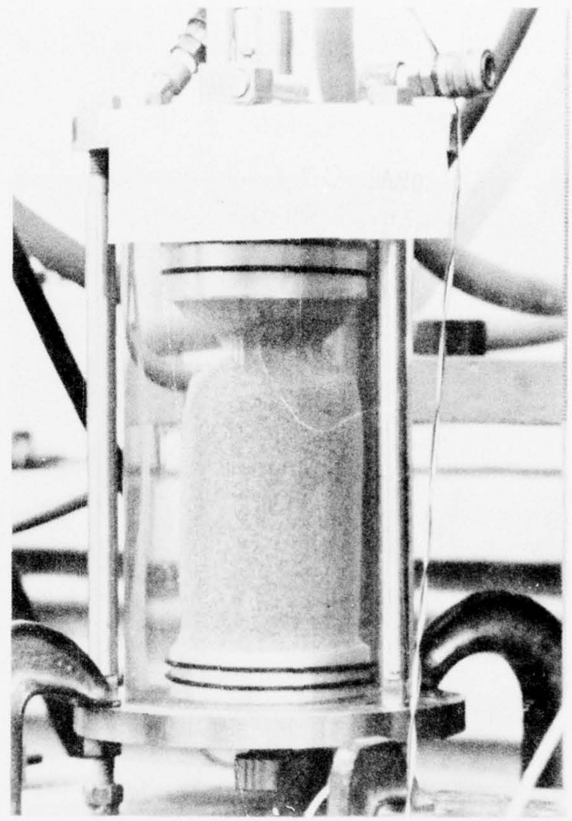
SAMPLE READY FOR TESTING

COMPRESSION

FIG. 2-31 SEQUENCE OF FAILURE FOR A SAMPLE OF DENSE MONTEREY SAND - REGULAR ENDS. $Dr = 90\%$



EXTENSION



EXTENSION

FIG. 2-31(Contd.) SEQUENCE OF FAILURE FOR A SAMPLE OF DENSE MONTEREY SAND - REGULAR ENDS. Dr = 90%

Effect of End Restraint on the Strength of Dense Sand (Dr = 80 Percent)

In order to obtain some conclusive results for dense Monterey sand, a new series of tests were conducted on samples at a lower relative density (Dr = 80 percent) so that failure could be induced using a lower level of cyclic stress. It should be noted that these samples at Dr = 80 percent did not neck or pull off the ends as did the samples at D_r = 90 percent.

The results of strength data for Dr = 80 percent samples with both frictionless and regular ends, at a cyclic frequency of 0.05 Hz, is shown on Figure 2-32. These results show that samples with frictionless ends were stronger by about 30 to 35 percent than samples with regular ends.

Effect of the Method of Sample Preparation on the Cyclic Strength of Monterey Sand

After completing the cyclic testing program for Monterey Sand, and comparing the results to those presented in the Mulilis (7), a question arose as to the method of sample preparation and its effect on the cyclic strength of Monterey sand. This question was brought about after noting that a recently completed study by Mulilis (7) found significant variations in cyclic strength of this Monterey No. 0 sand when specimens were prepared by different compaction techniques. Furthermore, Mulilis had presented one series of tests on loose Monterey No. 0 sand at 50 percent relative density and 8 psi consolidation pressure which indicated that lubricated ends gave almost the same results as regular ends (Fig. 2-33); with frictionless ends being slightly stronger.

The samples on which the Mulilis conclusions were based were

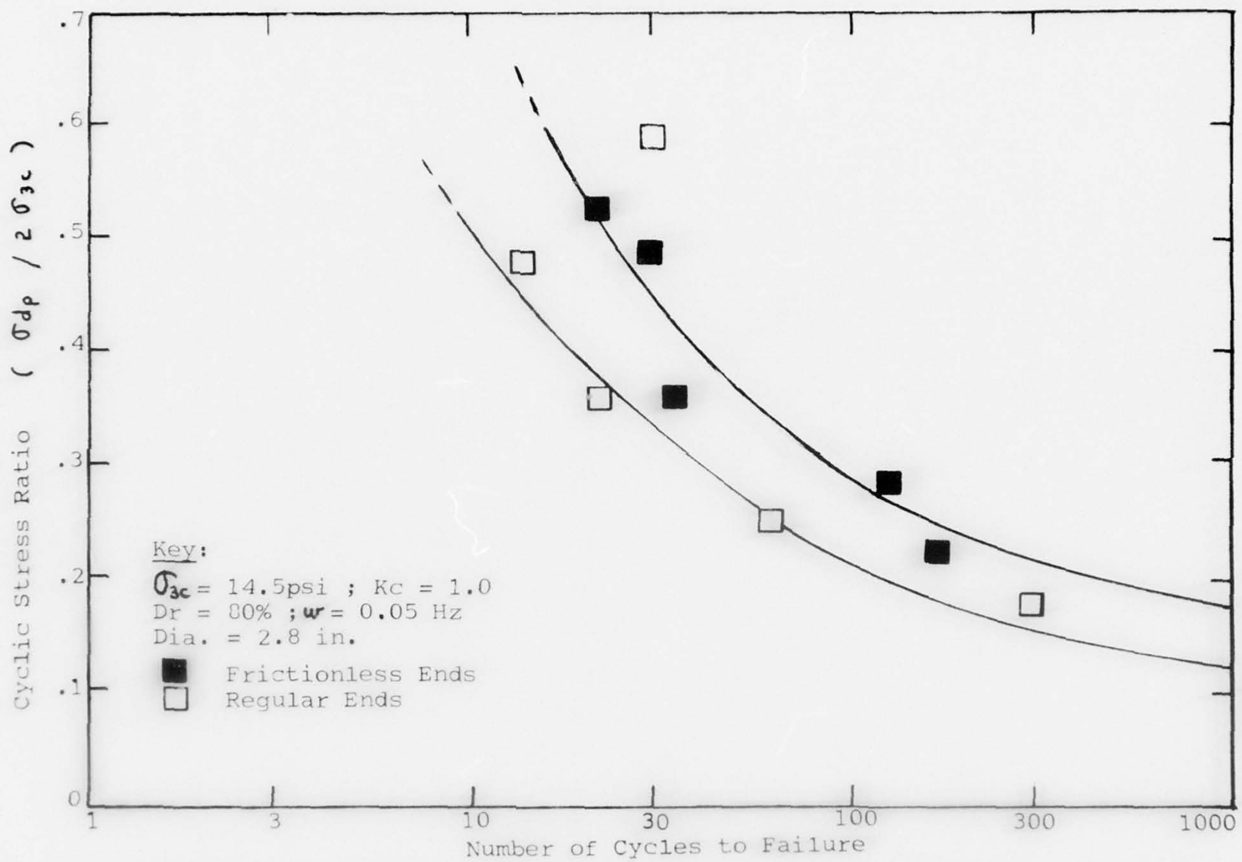


FIG. 2-32 COMPARISON OF STRENGTH DATA FROM TESTS ON DENSE MONTEREY SAND , Dr = 80%

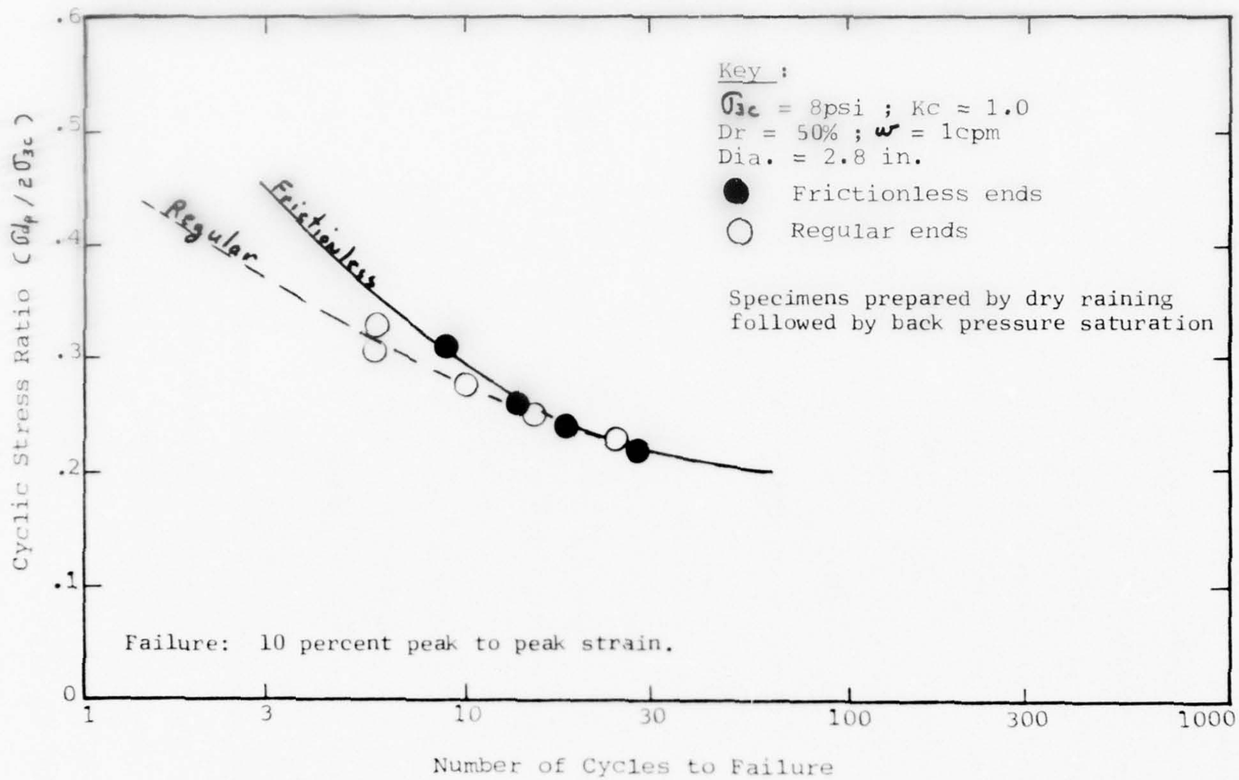


FIG. 2-33 STRENGTH DATA FROM CYCLIC TRIAXIAL TESTS ON LOOSE MONTEREY SAND - FROM MULILIS (7)

prepared by raining through air and then saturating. The tests performed for the study reported herein were performed on samples prepared by wet raining and vibrating to 60 percent relative density and tested at 14.5 psi confining pressure. Unfortunately, at the time these studies were being done, it was mistakenly thought that Mulilis had prepared his frictionless-regular comparative specimens by moist tamping, because this method was being spoken of favorably as probably the best method to prepare remolded samples of sand in the laboratory. Furthermore, Mulilis found that moist tamping led to specimens which were considerably stronger than samples prepared by wet or dry raining.

Therefore, under the mistaken opinion that the Mulilis comparative regular and frictionless tests were performed on moist compacted samples, a series of similar tests were also performed on Monterey No. 0 sand using a moist tamping preparation method. The results of this series of tests are shown in Figure 2-34 and indicate that the frictionless ends give about 10 percent higher strength than the regular ends. This is substantially the same order of magnitude of effect found by the earlier tests on samples prepared by wet raining. Unfortunately, the study was concluded before it was realized that these test specimens were not prepared in the same way as the Mulilis comparative frictionless-regular test specimens were prepared. It was then too late to go back and repeat the tests again using air raining and testing at 50 percent relative density.

Nevertheless, as will be discussed at the end of this report, the conclusions from these two studies may be completely compatible with each other. Overall, the data show that the effect of frictionless

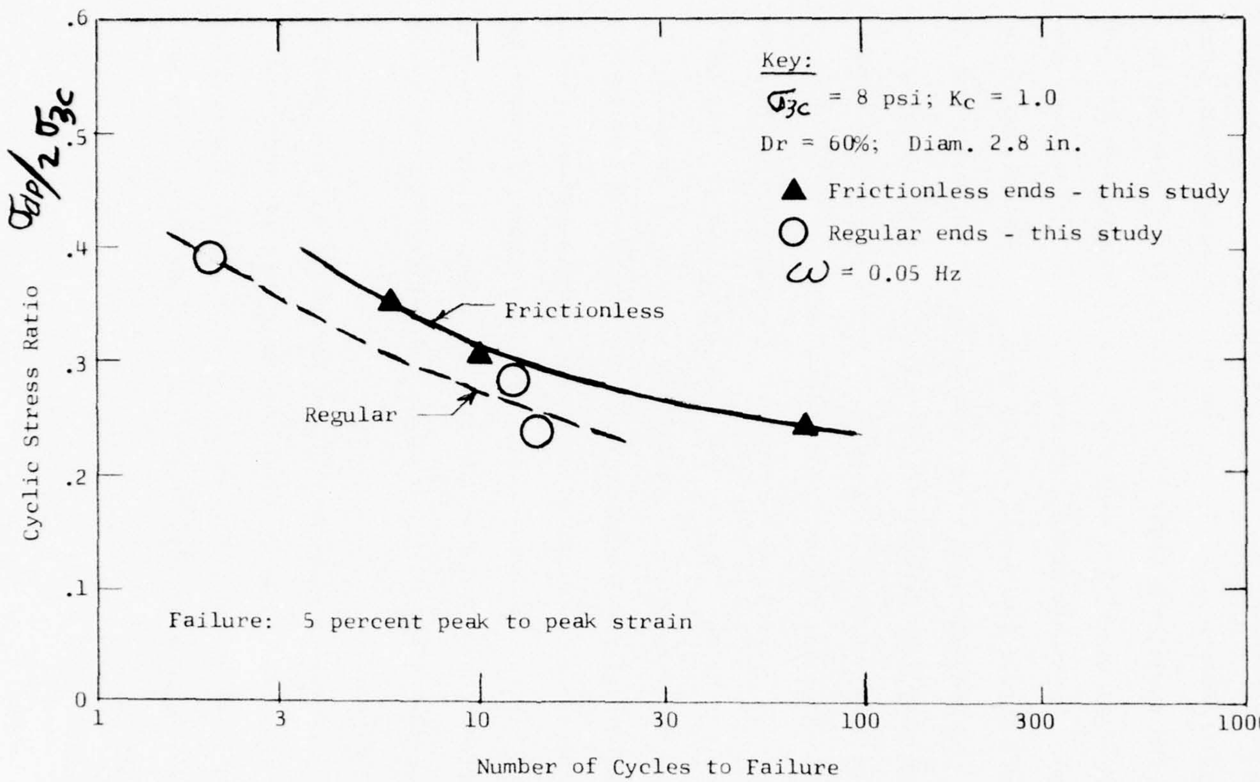


Fig. 2-34 COMPARISON OF STRENGTH DATA FROM CYCLIC TRIAXIAL TESTS ON LOOSE MONTEREY SAND

platens becomes progressively more important with increasing tendency for the soil to dilate under static monotonic loading. Thus, since the looser the sands, the less it will dilate, it follows that at Dr = 50% (Mulilis tests) the sand will dilate less than at Dr = 60% (this study). From this it follows that the frictionless ends should show less efects for the Mulilis tests than for the tests in these studies. Just how much less effect cannot be stated, but since this study showed only a 10 percent difference at Dr = 60%, it is not unreasonable that the Mulilis tests at Dr = 50% should show almost zero strength increase due to frictionless ends.

In addition to relative density, it is noted that the two studies also used different methods of compaction (Mulilis - air rain and this study - moist tamping). The Mulilis study gives detailed examples of how different methods of compaction have a very large effect on the cyclic strength of this sand. While not specifically investigated, it is reasonable that different methods of compaction may also affect the dilation tendency of this sand in such a way as to eliminate the effect of frictionless ends as an influence on the cyclic strength.

It is felt that further meaningful discussion on this specific topic will require a basis of additional special tests which not only address the frictionless end effects, but also address the particle structure arrangement and its significance to the many facets of granular soil behavior.

CHAPTER 3

EFFECT OF END RESTRAINT ON THE CYCLIC TRIAXIAL STRENGTH OF SACRAMENTO RIVER SAND

Sacramento River sand was used for many of the early studies on the cyclic strength of sand (4,8) and was used for the limited number of tests reported by Lee (2) in the previous report on the effect of frictionless ends.

This sand was prepared for testing by sieving between the No. 50 and No. 100 sieves. The end product which was tested was a fine, clean uniformly graded sand with the following properties: range of grain size 0.85 - 0.3 mm; D_{50} = 0.2 mm; e_{max} = 1.03; e_{min} = 0.61. A grain size curve is shown in Figure 3-1. Unfortunately only a small amount of this sand remained after the numerous previous testing programs. This small portion was used and reused several times in the tests performed for this study.

Behavior of Loose Sand

The samples in this series of tests were prepared by wet raining in the same manner as described for the Monterey Sand (see Fig. 1). It was also the same method used for all previous studies with this sand.

Figure 3-2 shows a comparison of the strength data from the Lee report (2) to strength data obtained in this study. Note that the results of this study are consistent with those reported by Lee (2), which demonstrated that these tests could be accurately duplicated for this later study.

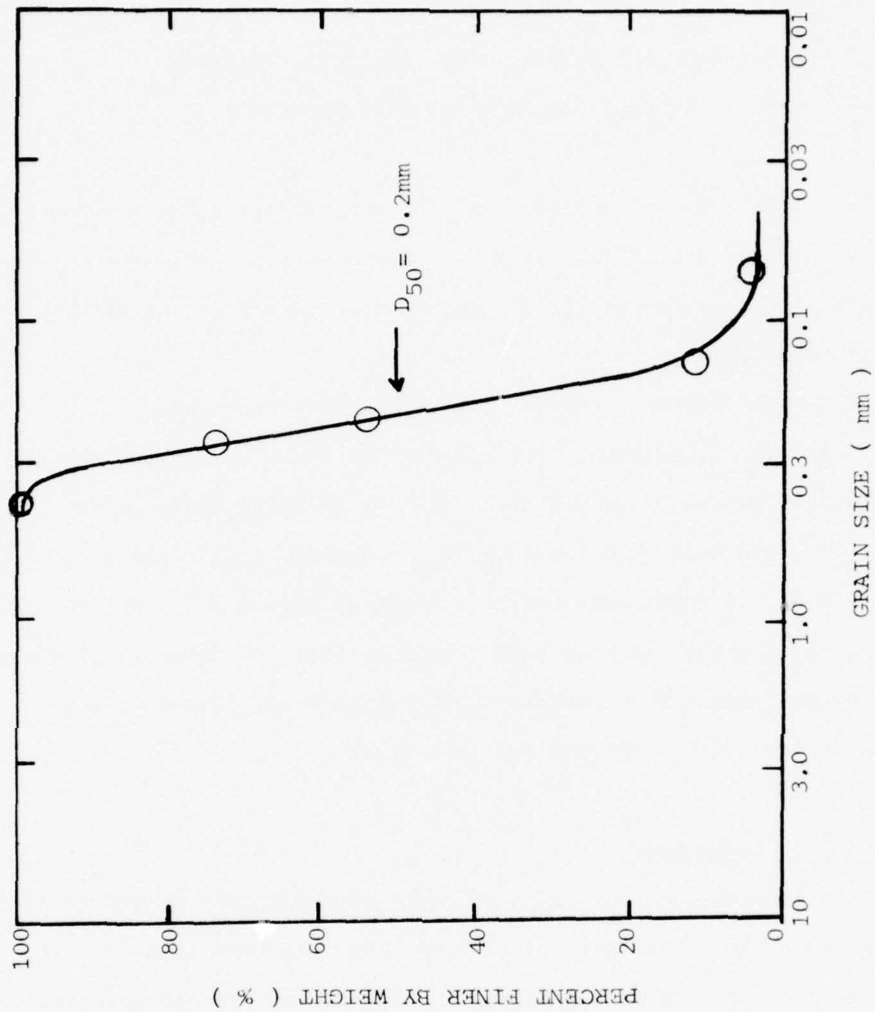


FIG. 3-1 GRAIN SIZE DISTRIBUTION CURVE FOR SACRAMENTO RIVER SAND

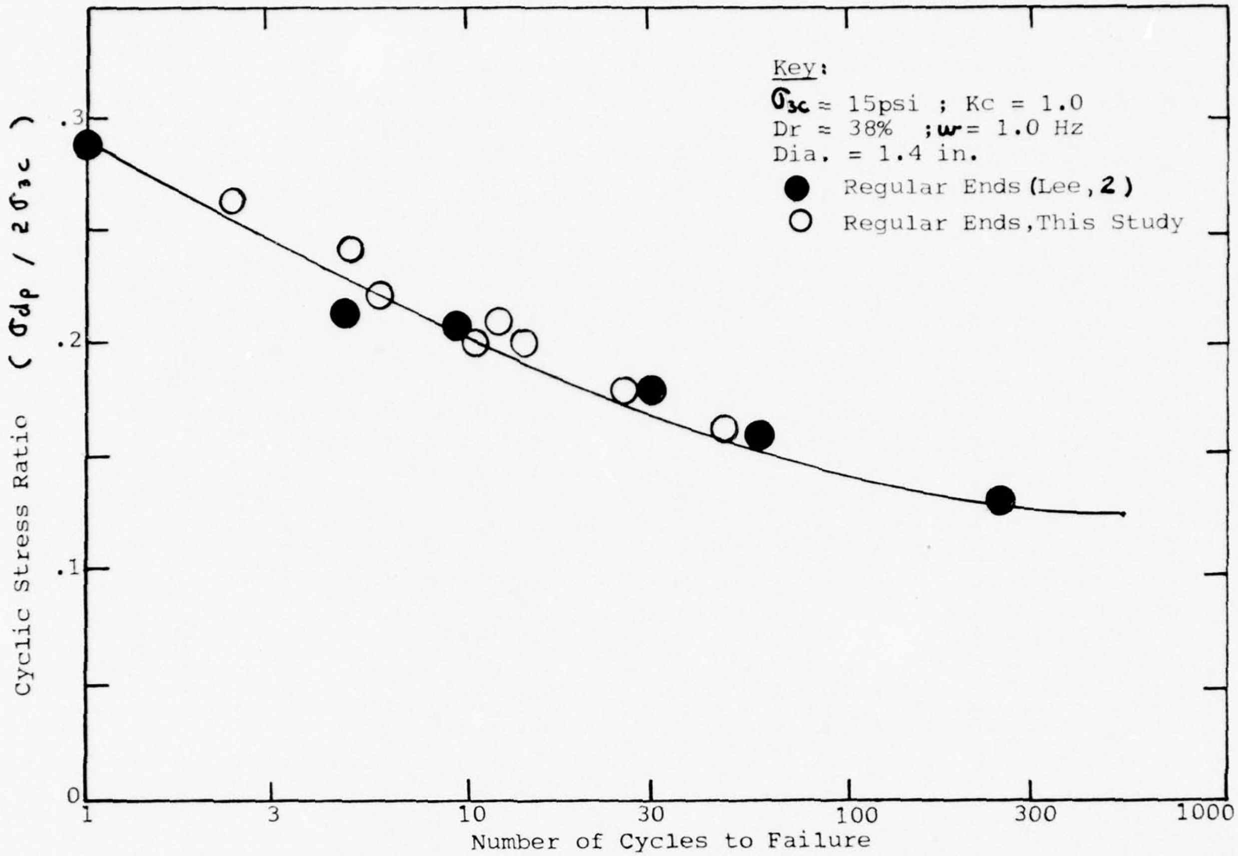


FIG. 3-2 COMPARISON OF STRENGTH DATA FROM TESTS ON LOOSE SACRAMENTO RIVER SAND

Effect of Prongs on the Strength of Loose Sand

As stated in Chapter 2, the frictionless end design used in this study had prongs in the center. Thus, the effect of these prongs had to be established to see if they had any significant influence on the cyclic strength of this soil. This was accomplished by running tests with regular ends without prongs and then comparing these results to tests on samples using regular ends with prongs.

The relationship between the cyclic stress and the number of cycles to cause failure for samples using regular ends with and without prongs is shown on Figure 3-3. A comparison of this data shows that prongs had no effect on the cyclic triaxial strength of loose Sacramento River sand. This observation is consistent with the results for Montery sand as shown on Figure 2-5.

Effect of End Restraint on the Strength of Loose Sand

Before the effect of end restraint could be determined, it was first necessary to investigate the effect of frequency on the cyclic strength of this sand. This was accomplished by running tests with frictionless ends at both 1.0 Hz and 0.05 Hz. The results of these tests are presented in Figure 3-4. Although they show somewhat more scatter than the tests results for Monterey sand, as shown on Figure 2-11, essentially they show the same trend, that cyclic frequency has no effect on the cyclic strength of the Sacramento River sand. Furthermore, the data obtained in this study are consistent with the results reported by Lee (2) for frictionless ends (Figure 3-5). Thus, the frequency at which a test is run does not effect the cyclic strength of Sacramento River sand whether using regular or frictionless ends.

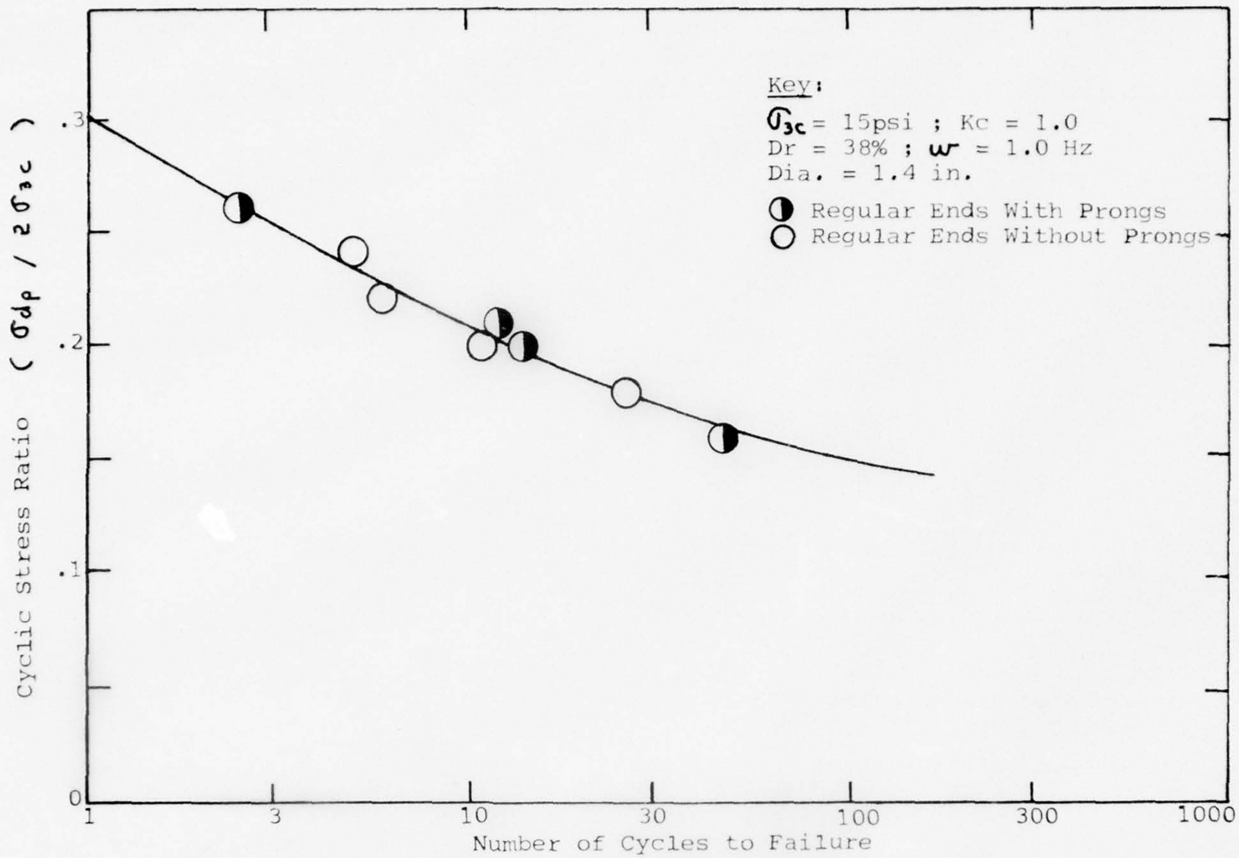


FIG. 3-3 EFFECT OF PRONGS ON THE CYCLIC STRENGTH OF LOOSE SACRAMENTO RIVER SAND

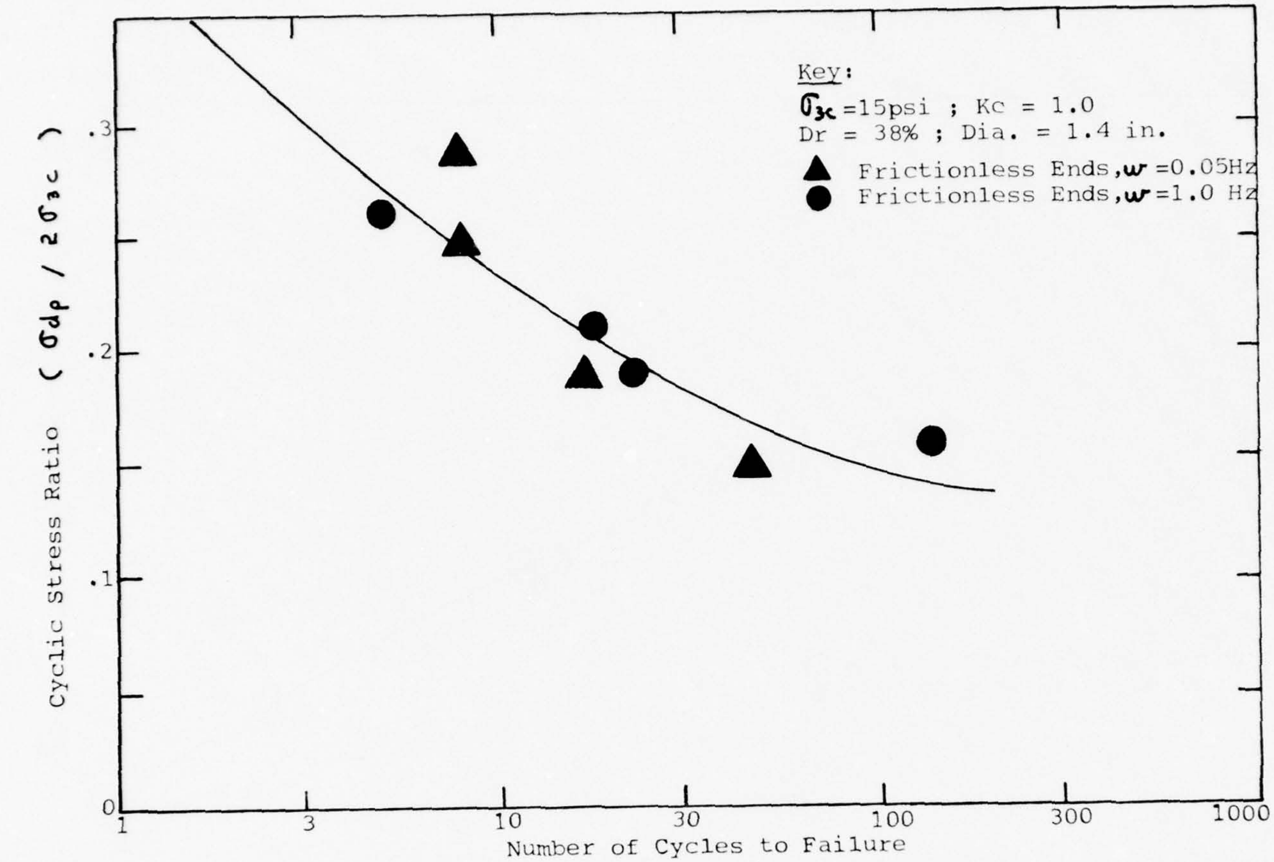


FIG. 3-4 EFFECT OF FREQUENCY ON THE CYCLIC STRENGTH OF LOOSE SACRAMENTO RIVER SAND

Figure 3-6 shows a comparison of the results of tests with frictionless ends to those run with regular ends on loose Sacramento River sand. This data shows that samples with frictionless ends had cyclic strengths about 10 to 15 percent stronger than samples with regular ends. Again, this is consistent with the results reported by Lee (2), as reproduced in Figure 3-7. A direct comparison between the data obtained by Lee (2) in the earlier study and that obtained in this study is shown in Figure 3-8. Note that the results from the two sets of tests are almost identical.

Behavior of Dense Sand

The samples of dense Sacramento River sand were prepared at a relative density of 80 percent. These tests were performed on samples with both regular and frictionless ends at cyclic frequencies of 0.05 Hz and 1.0 Hz. The results of these tests and the effect of prongs, frequency and end restraint on the cyclic strength of Sacramento River sand at 80 percent relative density are discussed below.

Effect of Prongs on the Strength of Dense Sand

Tests were conducted on samples of dense Sacramento River sand using regular ends with and without prongs to establish the effect of prongs on the cyclic strength of dense sand. Figure 3-9 shows a comparison of these results. As expected, from results of other studies described in earlier pages, prongs were found to have no effect on the cyclic strength of this dense Sacramento River sand.

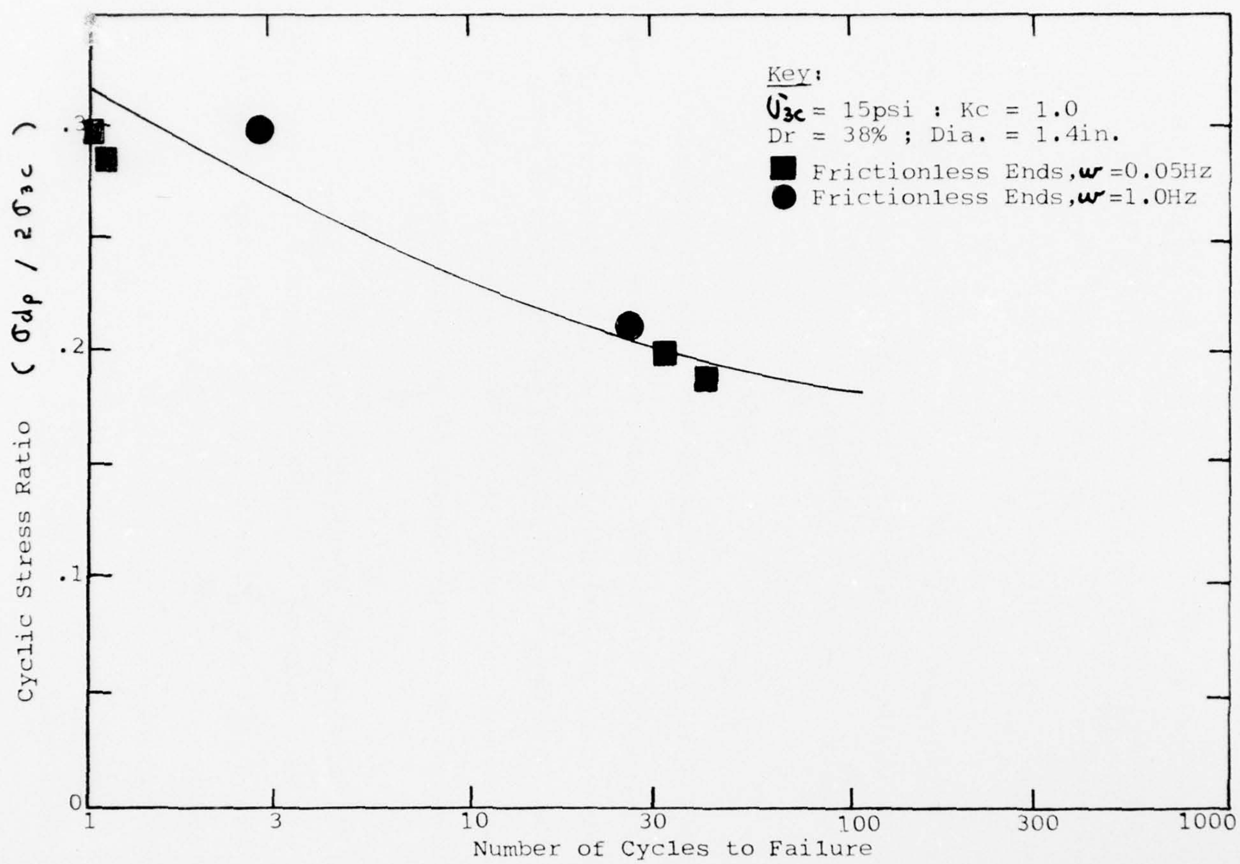


FIG 3-5 STRENGTH DATA FROM TESTS ON LOOSE SACRAMENTO RIVER SAND , LEE'S REPORT

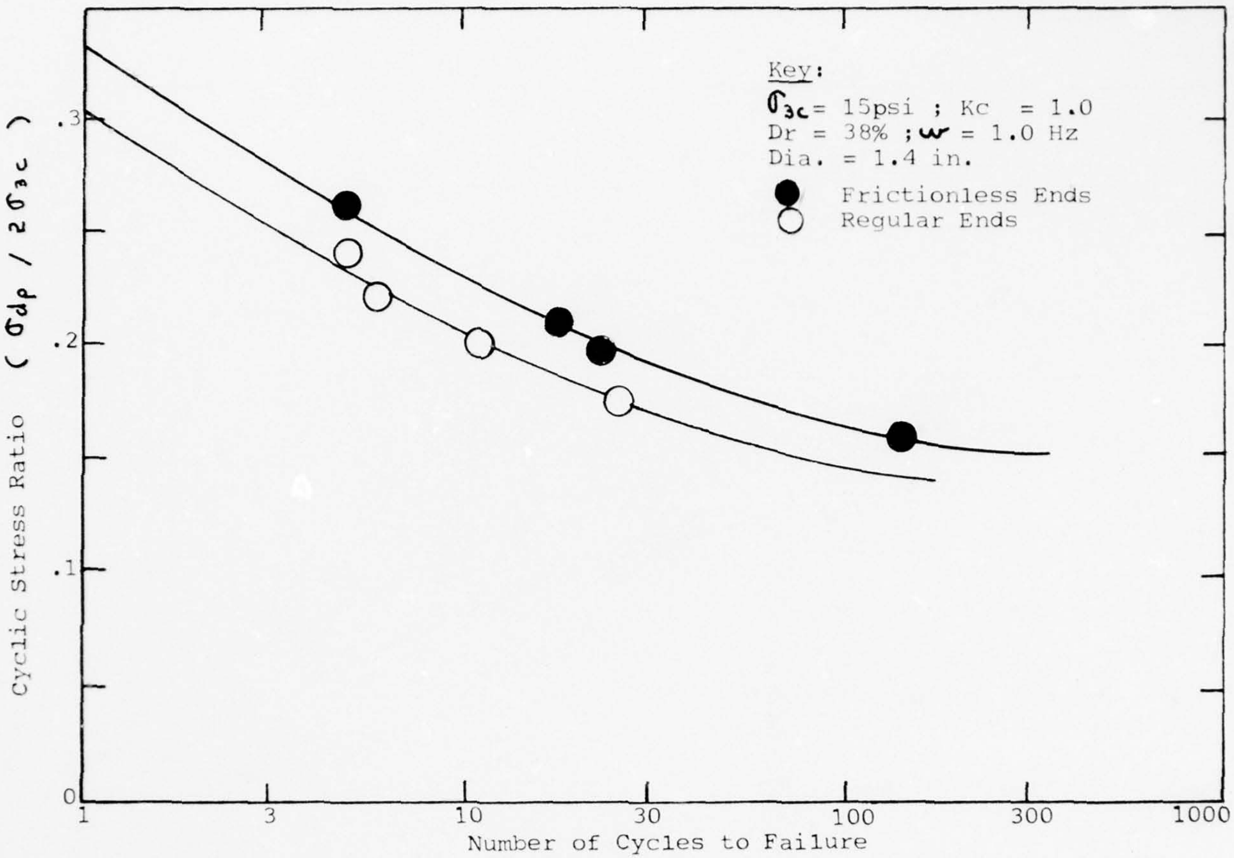


FIG. 3-6 COMPARISON OF STRENGTH DATA FROM TESTS ON LOOSE SCARAMENTO RIVER SAND

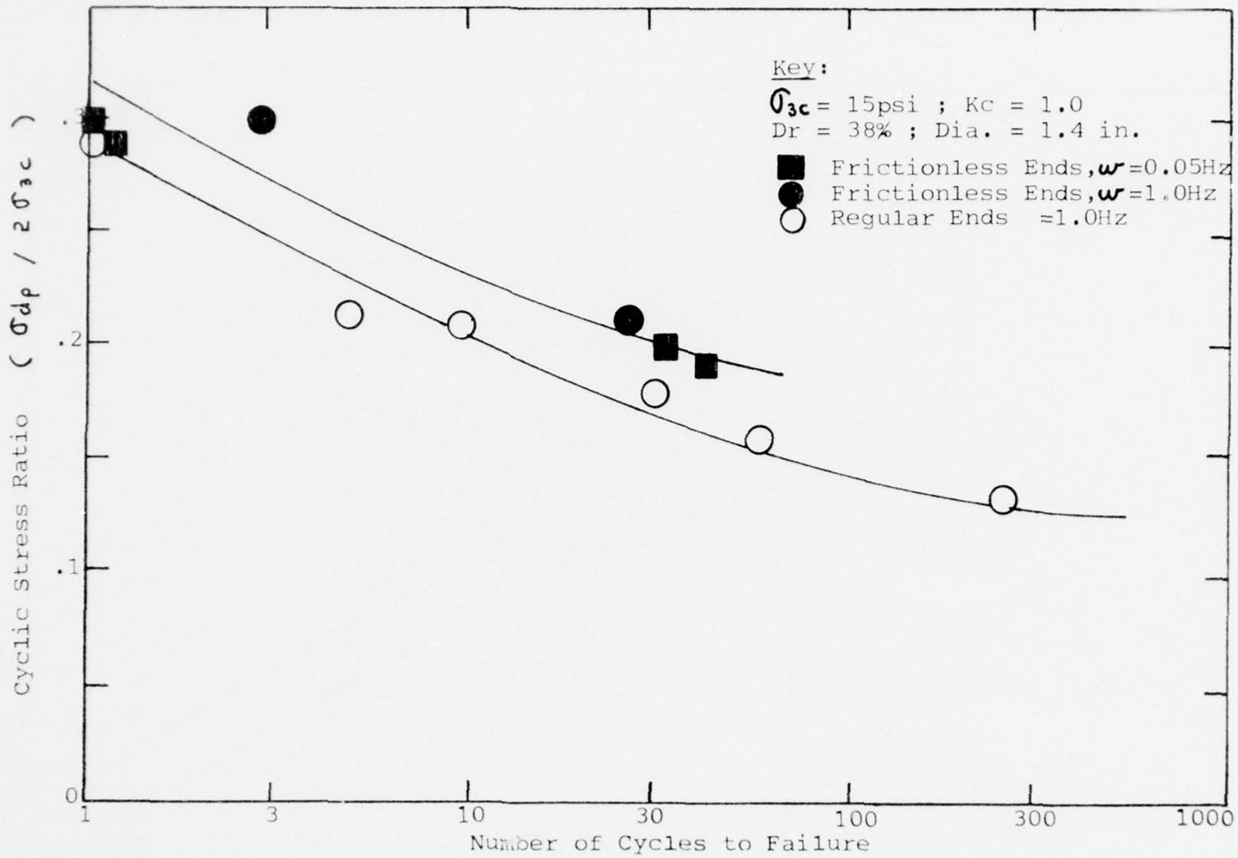


FIG. 3-7 COMPARISON OF STRENGTH FROM TESTS ON LOOSE SACRAMENTO RIVER SAND, LEE'S REPORT

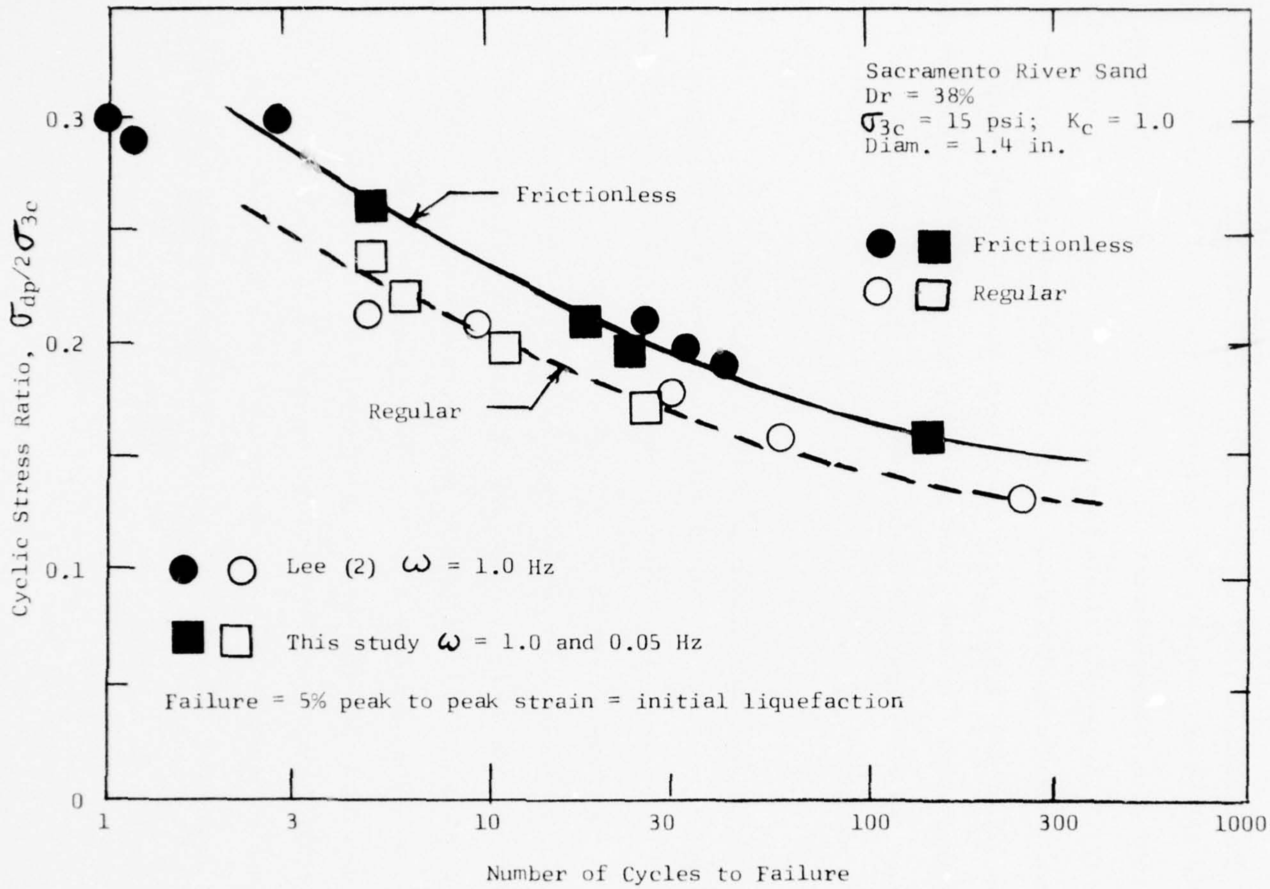


Fig. 3.8 SUMMARY COMPARISON OF CYCLIC STRENGTH FOR LOOSE SACRAMENTO RIVER SAND

Effect of Frequency on the Cyclic Strength of Dense Sand

The effect of frequency on the cyclic strength of Sacramento River sand at a relative density of 78 percent was already established by Lee (2). The results of his findings are reproduced on Figure 3-10. As shown, there is no difference in strength for tests run at cyclic frequencies of 1/6, 2 and 4 Hz. These tests were performed using a square load shape. However, they are consistent with the results shown on Figure 3-8 for tests on Monterey sand conducted at frequencies of 0.05 Hz and 1.0 Hz.

Effect of End Restraint on the Strength of Dense Sand

A series of tests on samples with frictionless ends was conducted in order to determine the effect of end restraint on the cyclic strength of dense Sacramento River sand. A comparison of these results with tests run on samples with regular ends is shown on Figure 3-11. The results show that using frictionless ends leads to an increase in strength of about 20 to 30 percent over tests with regular ends.

As a further supplement to the data obtained during this study, data reported by Lee (2) for cyclic triaxial tests on very dense (D_r = 100%) Sacramento River sand are reproduced in Figure 3-12. These data show an increase in cyclic strength of about 45 percent is obtained from using frictionless ends as compared with data using regular ends.

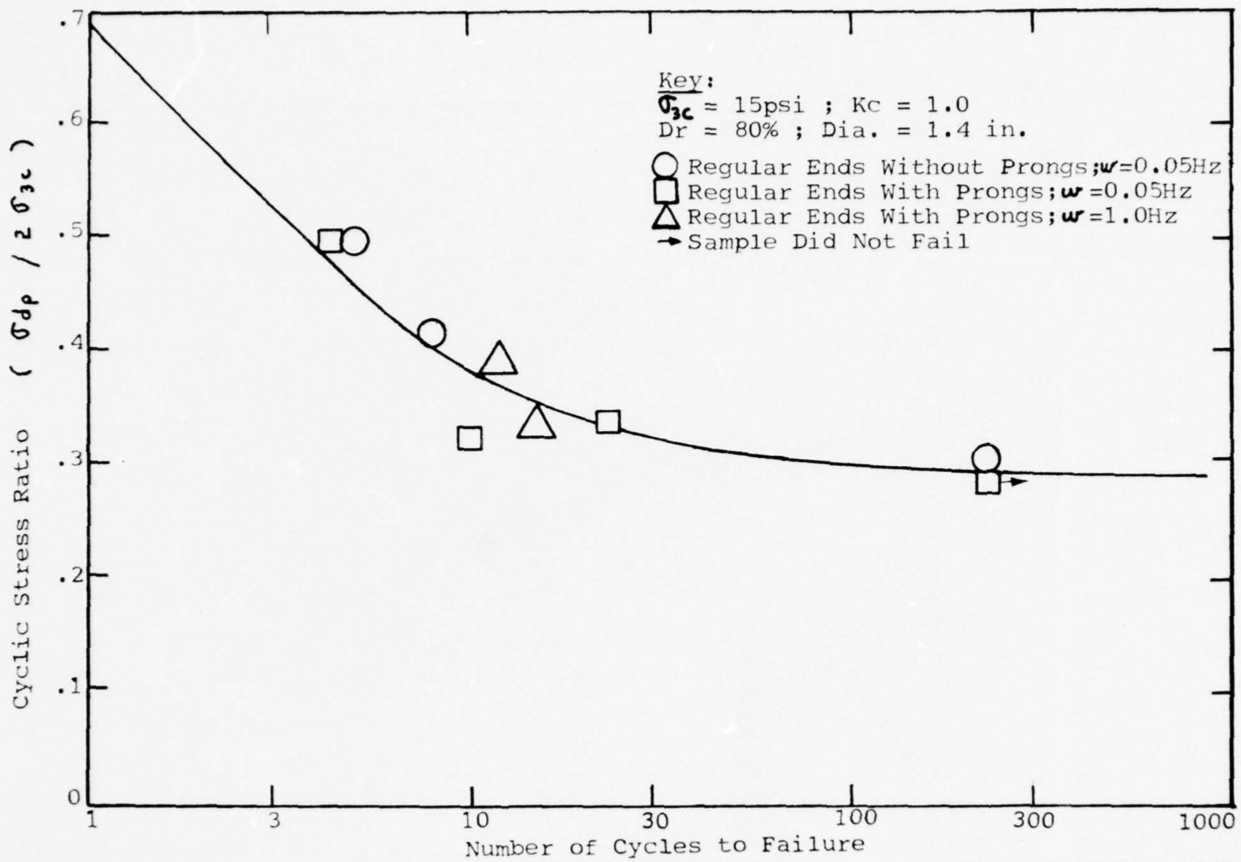


FIG.3-9 EFFECT OF PRONGS AND FREQUENCY ON THE CYCLIC STRENGTH OF DENSE SACRAMENTO RIVER SAND

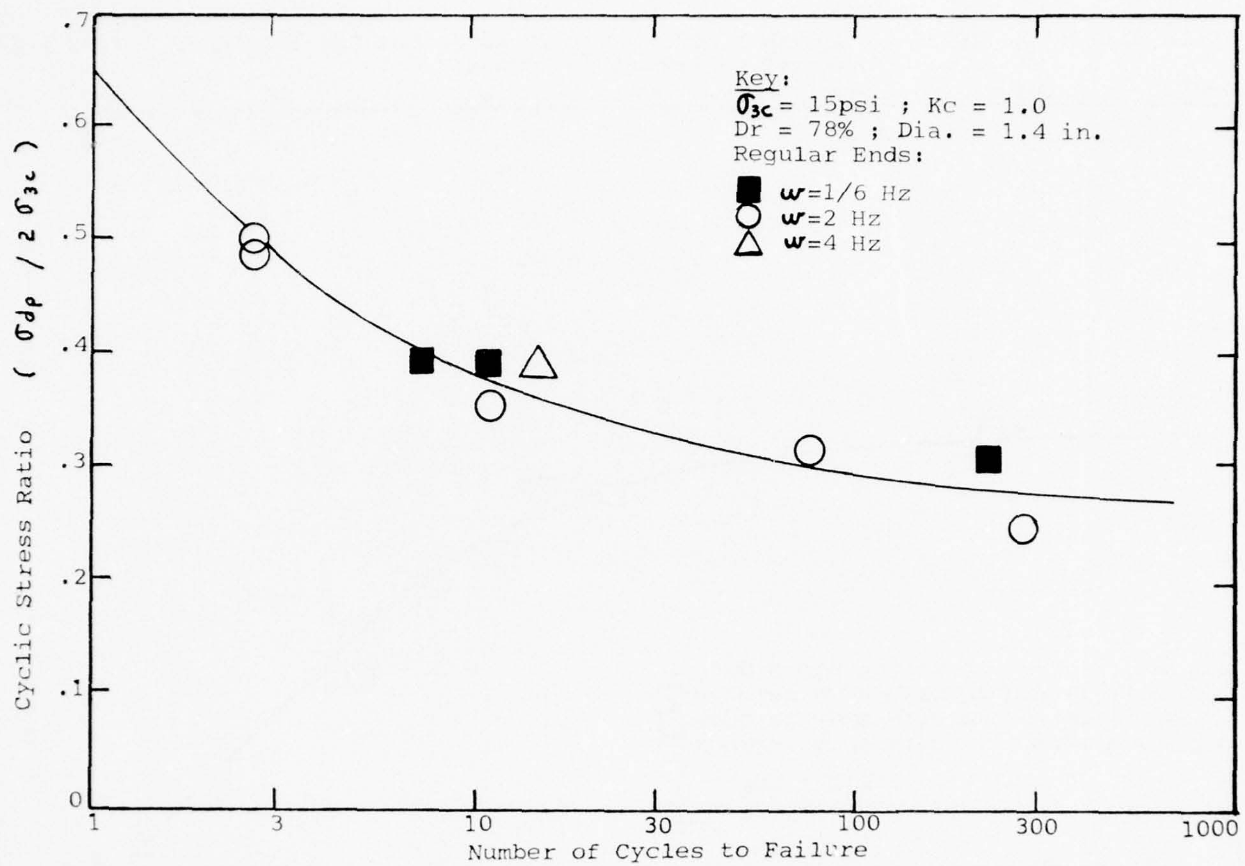


FIG. 3-10 EFFECT OF FREQUENCY ON THE STRENGTH OF DENSE SACRAMENTO RIVER SAND, LEE (2)

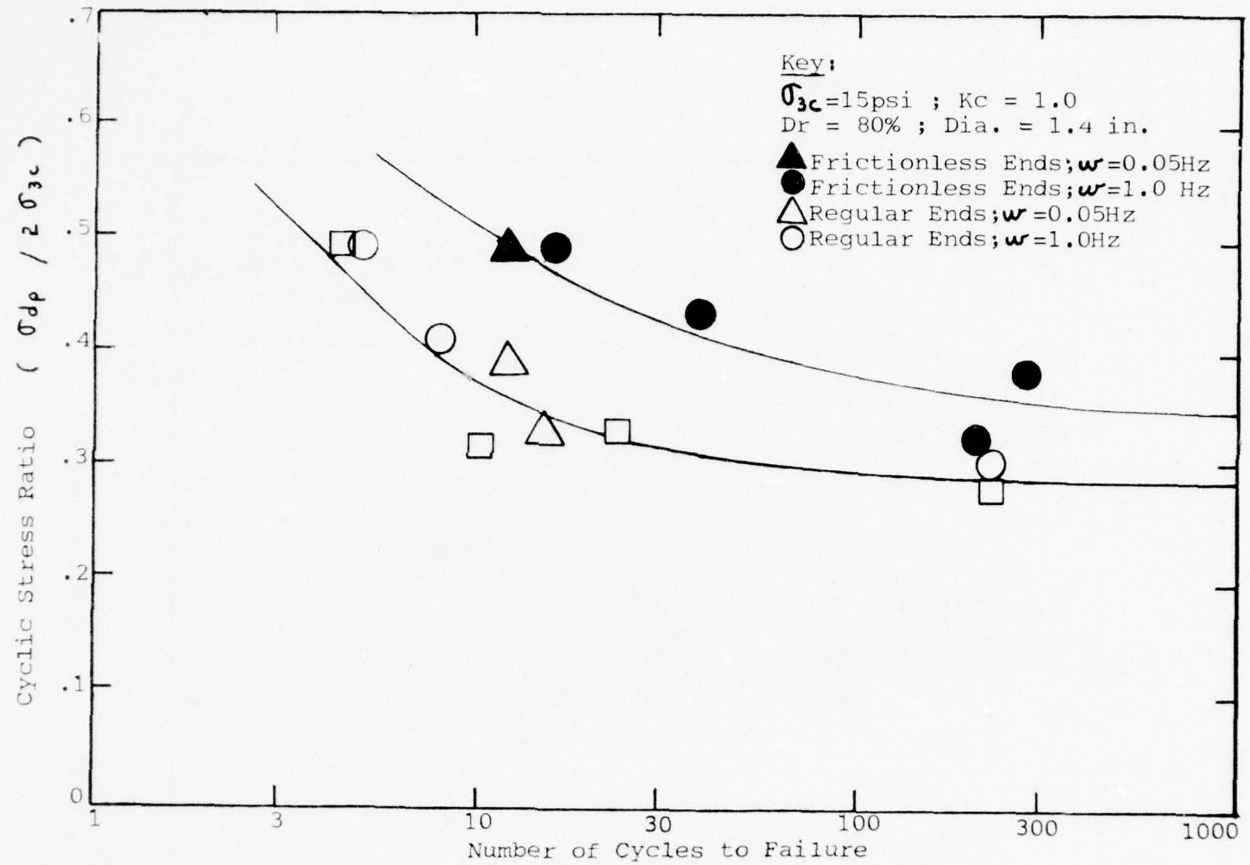


FIG. 3-11 COMPARISON OF STRENGTH DATA FROM TESTS ON DENSE SACRAMENTO RIVER SAND

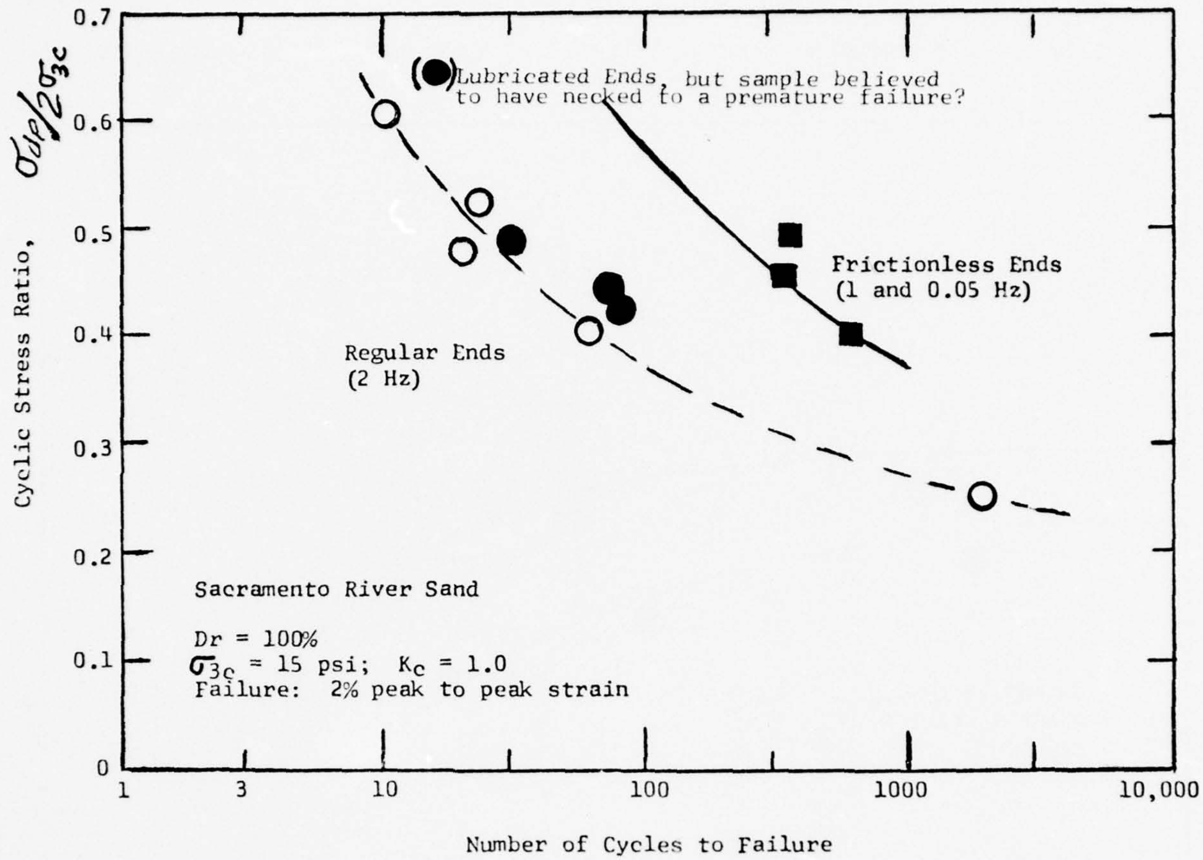


Fig. 3-12 COMPARISON OF CYCLIC STRENGTH DATA FROM TESTS ON VERY DENSE SACRAMENTO RIVER SAND (From Lee - 2)

CHAPTER 4

EFFECT OF END RESTRAINT ON THE CYCLIC TRIAXIAL STRENGTH OF L.A. HARBOR SAND

L. A. Harbor sand is a silty fine sand dredged from the Los Angeles Harbor area and used as hydraulic fill to expand the Harbor facilities. The grain size curve for the sample of this soil used in these studies is presented in Figure 4-1. Additional soil characteristics and previous cyclic testing data have been reported by Knuppel (1).

Cyclic triaxial tests, were performed on this soil at a relative density of 60 percent using a confining pressure of 15 psi. The samples were prepared and tested in the same wet raining manner as described previously for other soils already presented. Because the soil contained about 2 percent silt, a two stage procedure was used in an effort to maintain the same silt content even though it did not readily rain out of the water with the sand. The silt remaining in the flask after the sand was poured out was saved. Much of the water was removed by evaporation, and then the remaining high silt concentrated water was poured into the sample mold before raining in the sand for the next sample.

Effect of Prongs

For this sand, it was also deemed necessary to investigate the possible effect of prongs and cyclic loading frequency. Thus, regular ended samples of L. A. Harbor sand were tested with and without prongs at five frequencies. The results are shown on Figure 4-2. There was no observed effect of prongs or frequency on the cyclic strength

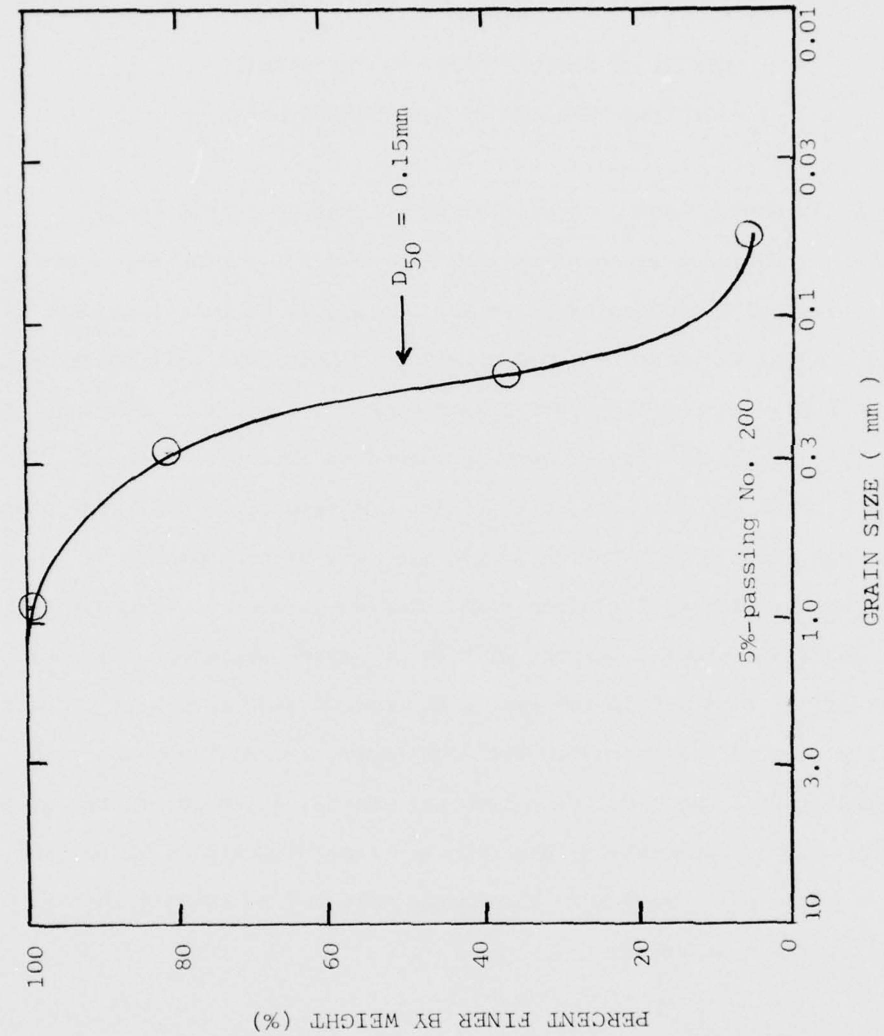


FIG. 4-1 GRAIN SIZE DISTRIBUTION CURVE FOR L.A.HARBOR SAND

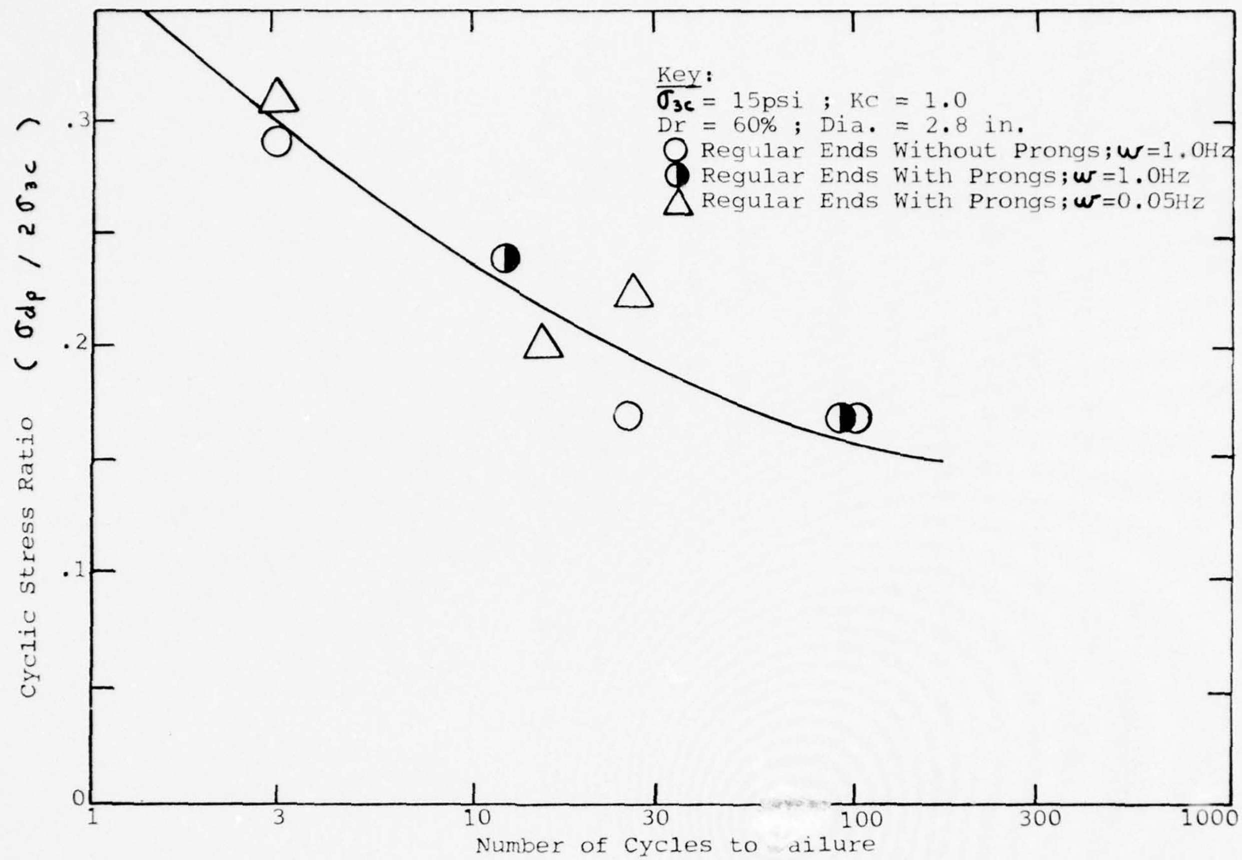


FIG. 4-2 EFFECT OF PRONGS AND FREQUENCY ON THE CYCLIC STRENGTH OF L.A. HARBOR SAND

of this sand.

Effect of End Restaint

Both regular and frictionless ends were used in testing samples of L. A. Harbor soil. Comparisons of these results are shown on Figures 4-3 and 4-4 for both 1.0 Hz and 0.05 Hz respectively. The results show that at both high and low frequencies there is no effect of end restraint. Thus, using either frictionless ends or samples with regular ends does not change the cyclic strength of L. A. Harbor sand.

A summary comparison of all cyclic test data on the L. A. Harbor sand is presented in Figure 4-5 in such as way as to illustrate the effect of cyclic loading frequency. Note that within the range of experimental error for this soil, there appears to be no difference between tests conducted at 1.0 Hz and those conducted at 0.05 Hz.

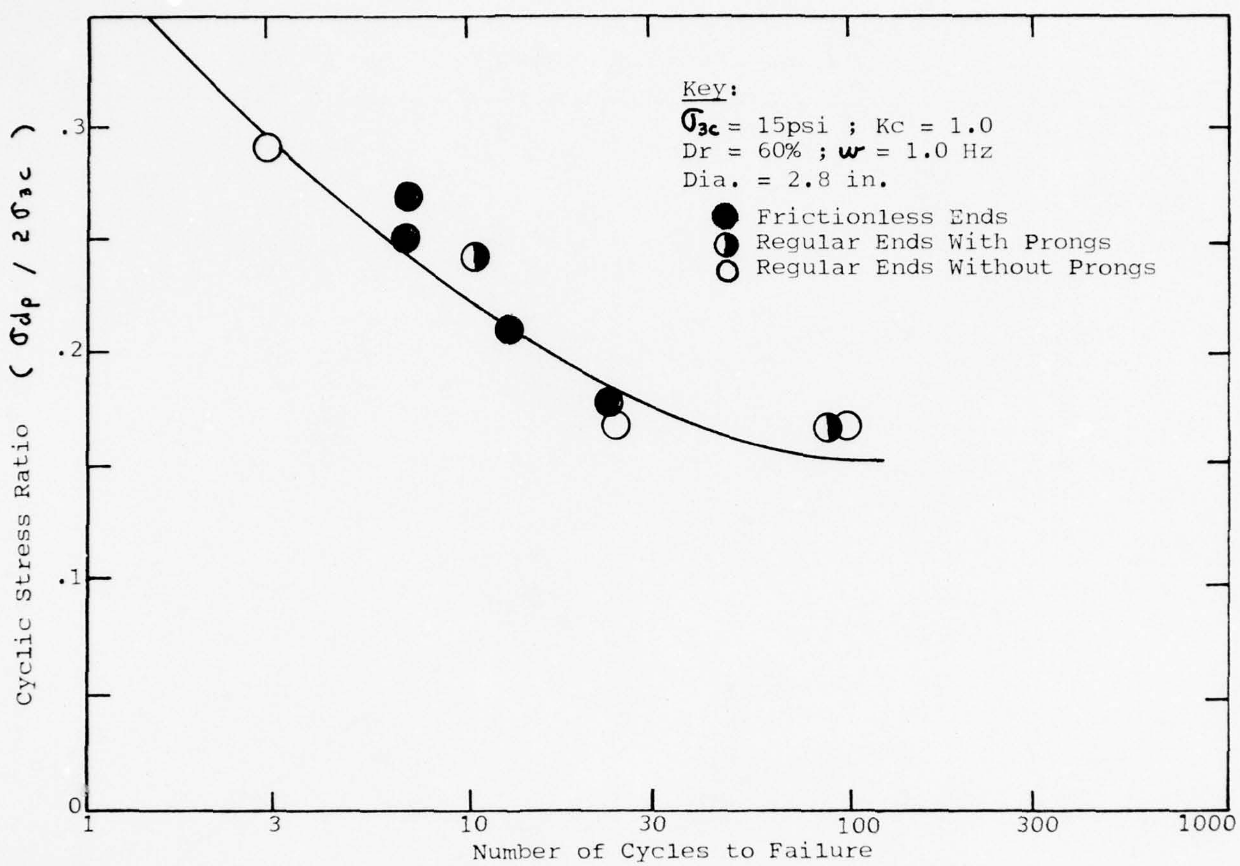


FIG. 4-3 COMPARISON OF STRENGTH DATA FROM TESTS ON L.A. HARBOR SAND

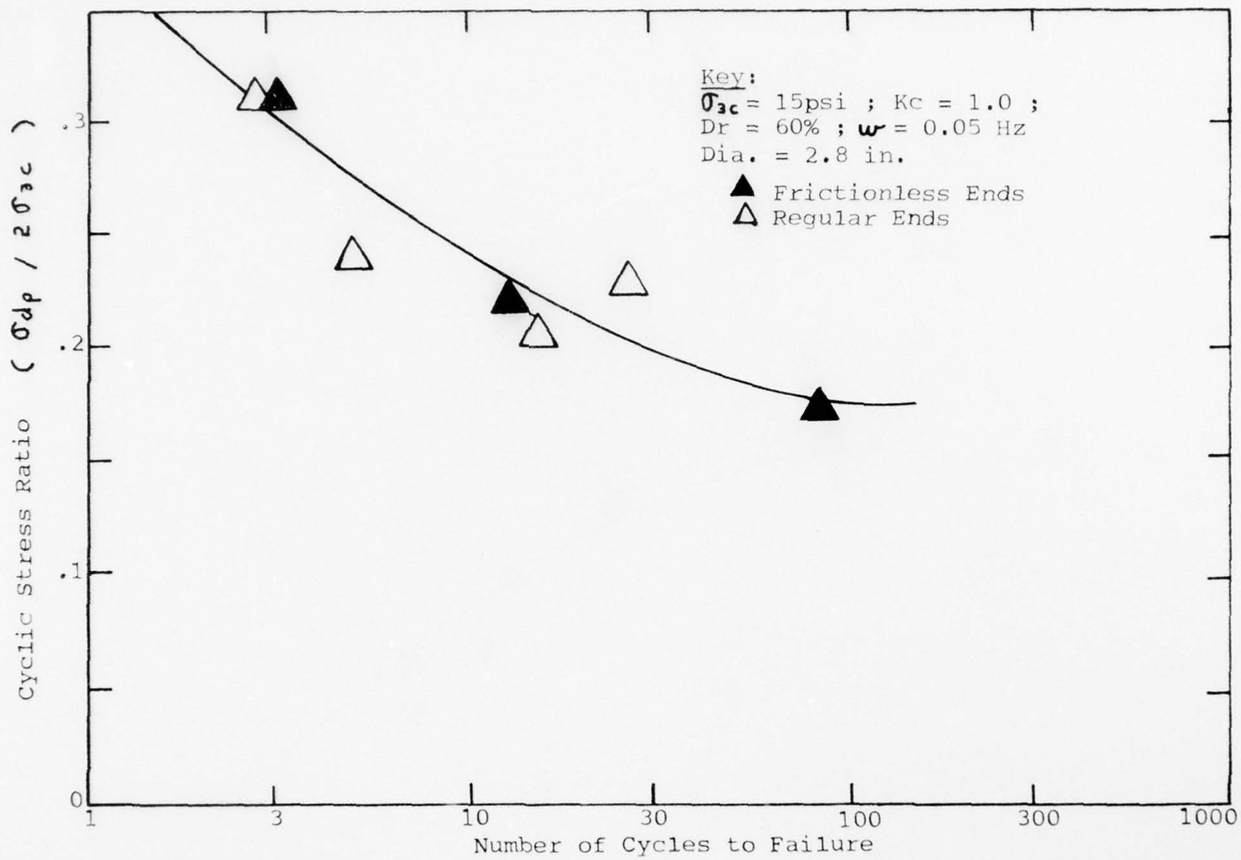


FIG. 4-4 COMPARISON OF STRENGTH DATA FROM TESTS ON L.A. HARBOR SAND.

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EFFECT OF FRICTIONLESS CAPS AND BASES IN THE CYCLIC TRIAXIAL TE--ETC(U)
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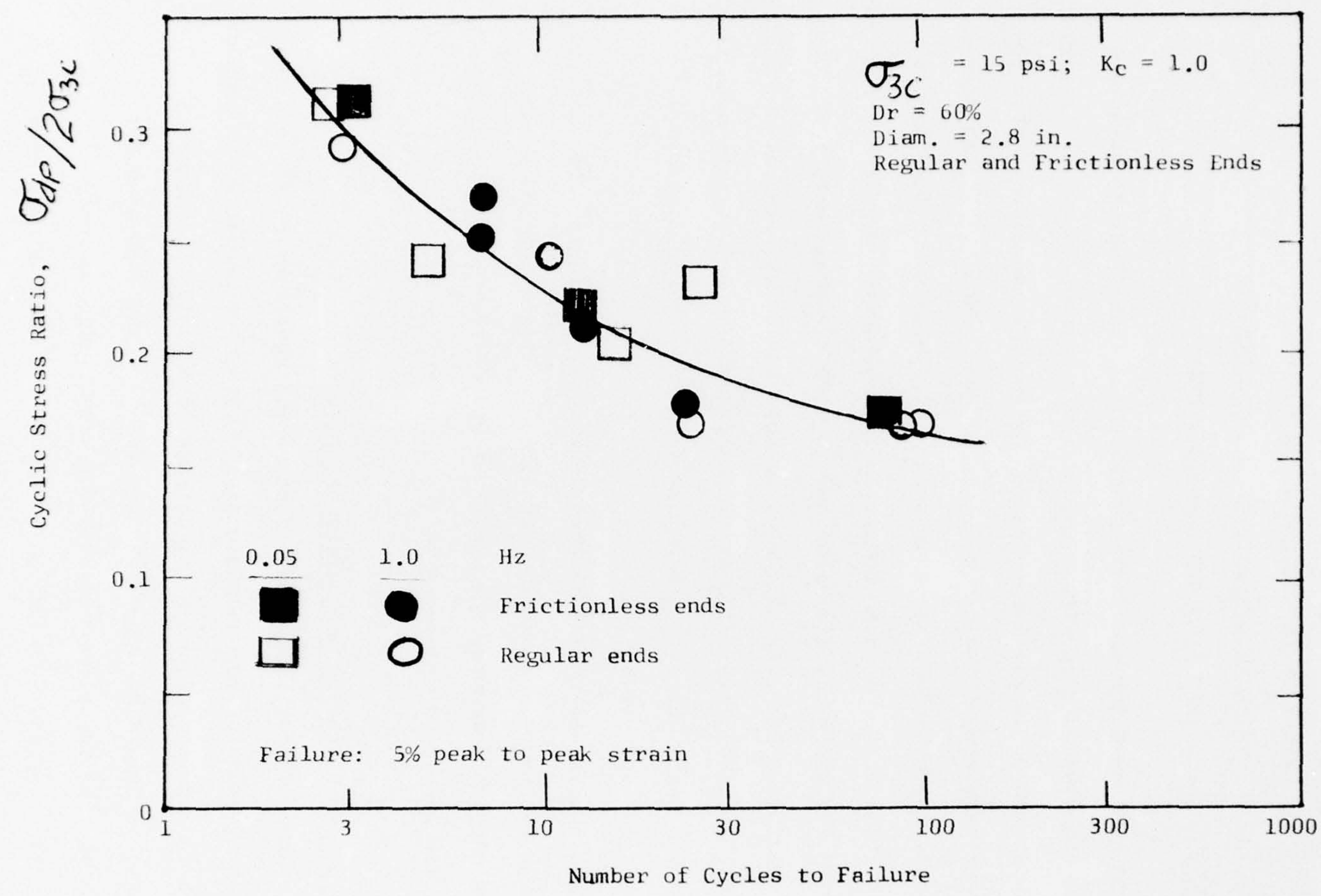

Fig. 4-5 Effect of Loading Frequency on the Cyclic Strength of L. A. Harbor Sand

CHAPTER 5

EFFECT OF END RESTRAINT ON THE CYCLIC TRIAXIAL STRENGTH OF UNDISTURBED AND REMOLDED CHAMPLAIN CLAY

The soil used for the tests described in this chapter was a very brittle undisturbed clay with a liquid limit between 25 and 27; plastic limit = 18 and a natural water content between 22 and 23 percent. The soil came from the St. Lawrence Valley, Quebec, Canada. It had been shipped to the UCLA laboratory in undisturbed blocks for some special tests. These tests had been completed several months previous. The tests performed for the study described herein used the small amount of soil which remained. The clay had hardened somewhat during storage and became quite stiff and brittle. The static undrained compressive strength from UU triaxial tests was q_u = 110 psi at the time the cyclic tests were performed for this study.

Cyclic triaxial unconsolidated-undrained (UU) tests were conducted on both undisturbed and remolded specimens, using frictionless and regular ends to determine the effect of end restraint on this Champlain clay. The results of these tests are presented below.

Effect of End Restraint on Undisturbed Clay

In order to determine the effect of end restraint on the cyclic strength of undisturbed Champlain clay, tests were conducted on samples using both regular and frictionless ends at a cyclic frequency of 0.05 Hz. It should be noted that the frictionless ends used for samples of this Champlain Clay did not have prongs in their centers. Because the clay was very brittle, the samples would always crack at the ends when the prongs were inserted. Fortunately, it turned out

that even without prongs the samples did not slide to one side even on the greased platens made with two layers of rubber each 0.012 in. thick and each separated by a generous smear of high vacuum silicone grease. In addition, no drainage was provided in the end caps. All tests performed on this clay were of the unconsolidated-undrained (UU) type. A total cell pressure of 60 psi was used for all tests in the series.

The results of a typical cyclic triaxial test, using regular ends, on an undisturbed sample of Champlain clay are shown on Figure 5-1. The sample strained at a slow even rate until it reached failure. Because of the undisturbed brittle nature of this clay, failure was taken as 3 percent double amplitude strain. For this test, failure occurred on the 15th cycle.

The relationship between the cyclic stress and the number of cycles to cause failure for samples us[illegible] regular and frictionless ends is shown on Figure 5-2. Comparing [illegible] results shows that there is no difference in strength gained by using frictionless ends or regular ends.

Effect of End Restraint on Remolded Clay

In order to study the effect of end restraint on remolded clay, cyclic triaxial tests were performed on compacted samples using frictionless and regular ends for remolded samples. The results of a Standard Proctor compaction test are shown in Figure 5-3. The remolded samples were prepared by compacting to a dry density of 117 lb/ft^3 (98% of Standard Proctor maximum) at a water content of 15 percent (on the wet leg of the compaction curve). The static undrained compressive strength from UU tests at 60 psi total confining pressure was about $q_u = 60$ psi.

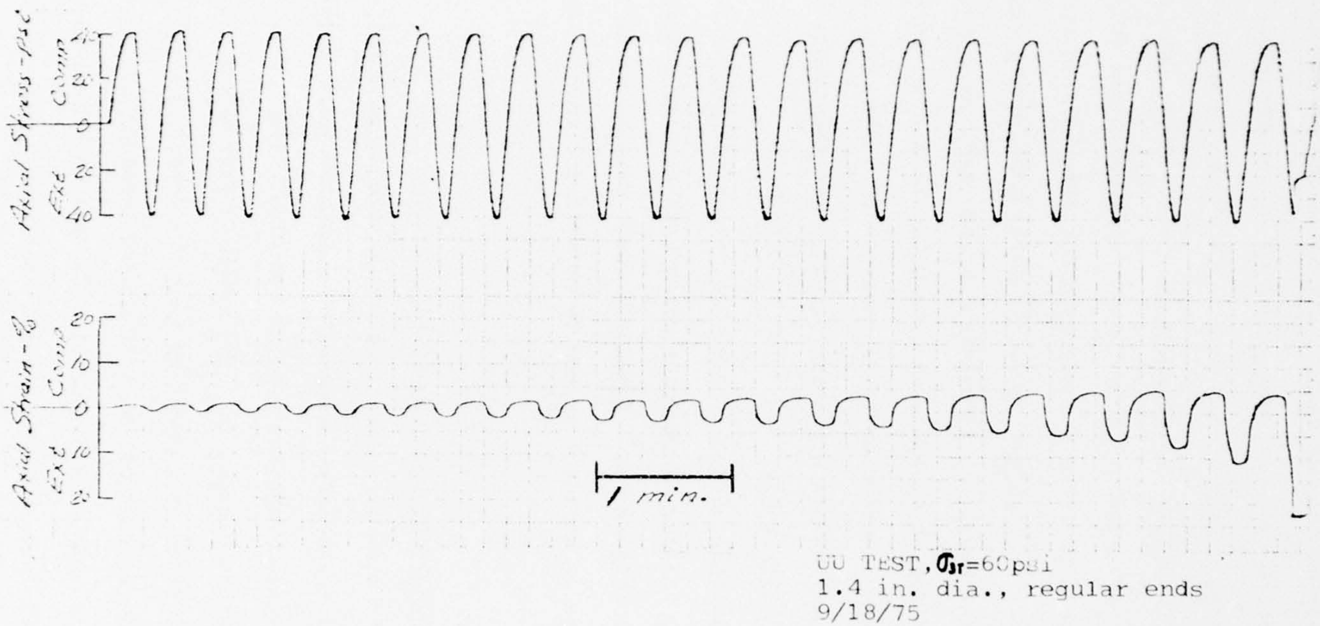

FIG. 5-1 TYPICAL RECORD OF CYCLIC TRIAXIAL TEST ON UNDISTURBED CHAMPLAIN CLAY

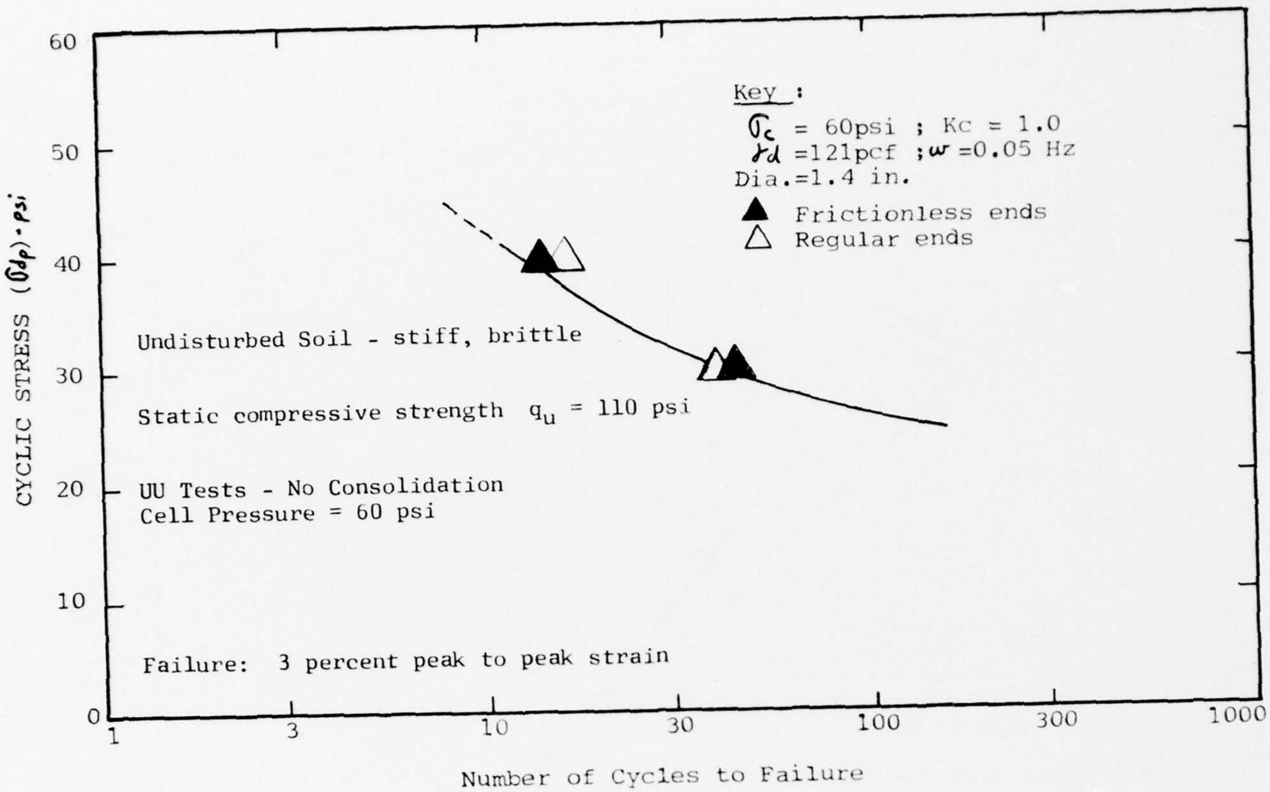

FIG. 5-2 COMPARISON OF STRENGTH DATA FROM CYCLIC TRIAXIAL TESTS ON CHAMPLAIN CLAY-UNDISTURBED

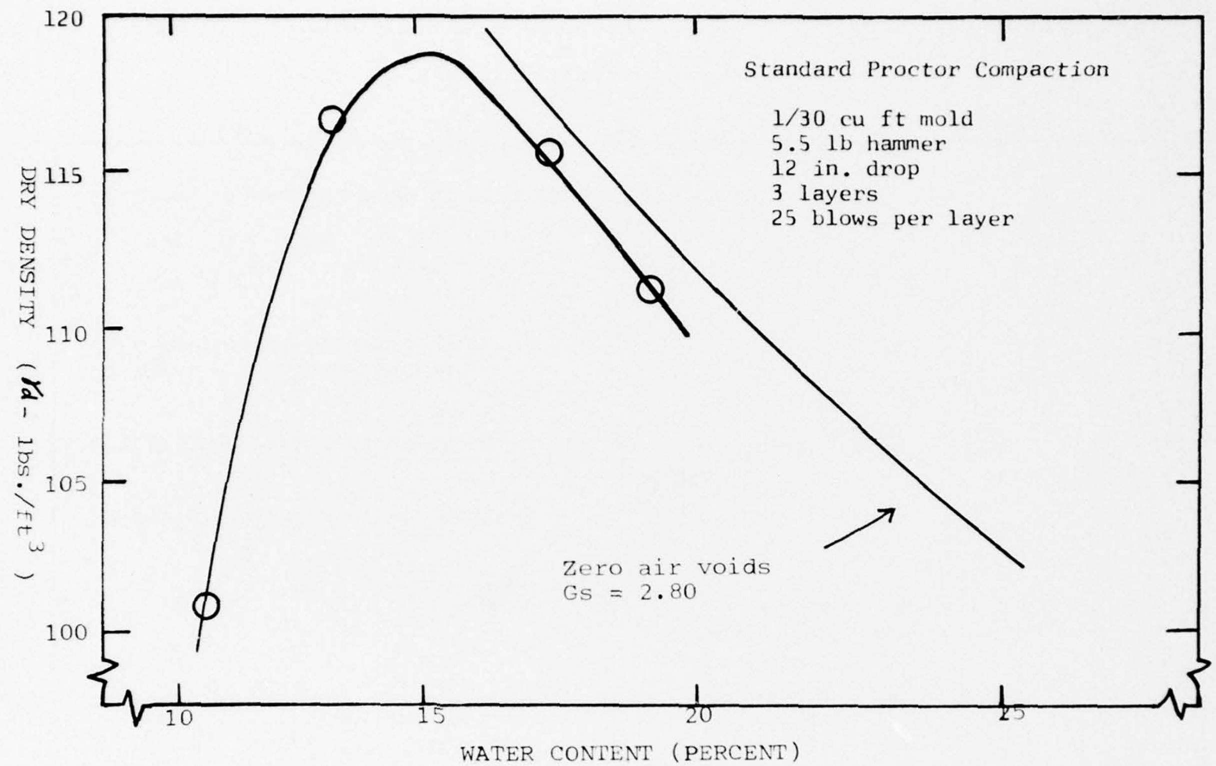

FIG. 5-3 STANDARD PROCTOR COMPACTION TEST ON CHAMPLAIN CLAY, A-SOIL

As with the undisturbed samples, the frictionless ends used for the remolded samples also did not have prongs in their centers, and the tests were performed as UU tests. There was no drainage, hence no backpressuring. The samples were therefore not completely saturated although they must have been nearly saturated because they were compacted wet of optimum on the wet leg of the compaction curve, and they were compressed to a total cell pressure of 60 psi. To be consistent with the tests on undisturbed samples, failure was defined in terms of 3 percent double amplitude strain, although the compacted samples were not brittle and could be readily strained to large amounts.

The cyclic strength data presented on Figure 5-4 indicates no distinguishable effect between the results with regular and with frictionless ends. Thus, the type of platen does not influence the cyclic strength of remolded or undisturbed Champlain clay.

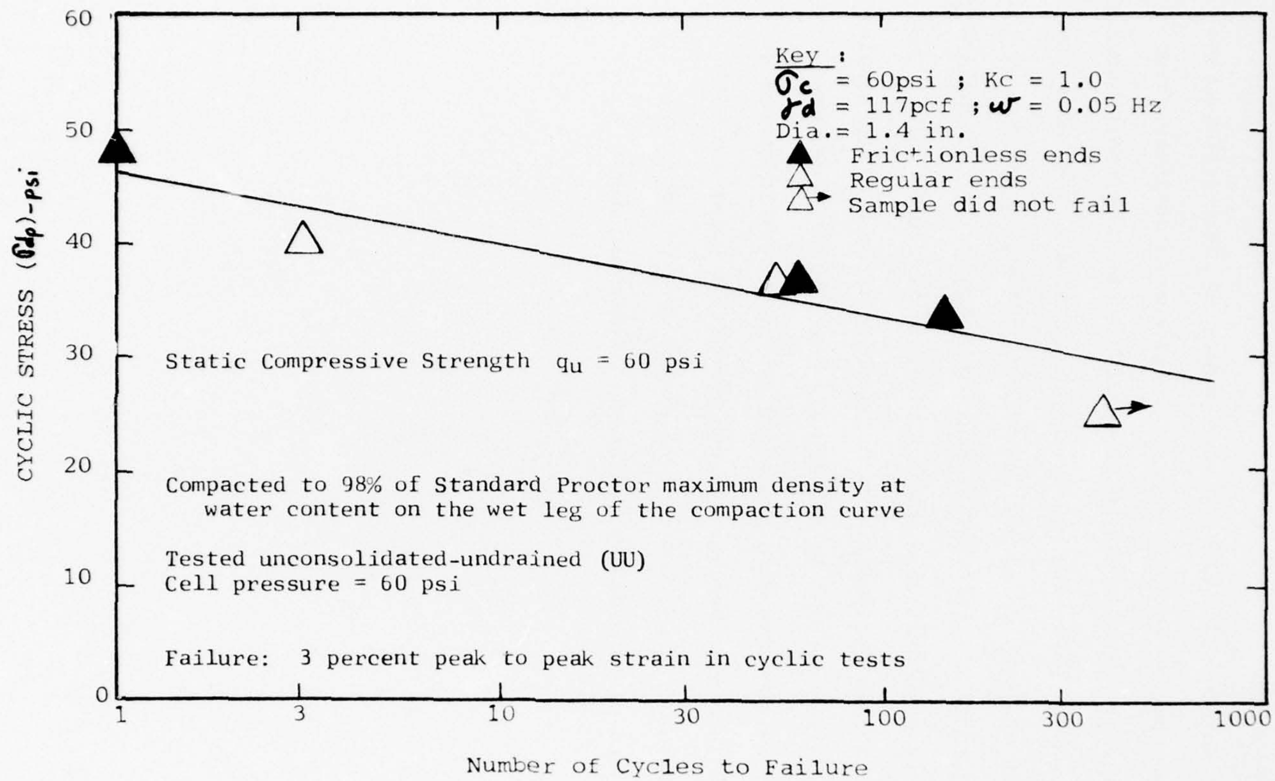

FIG. 5-4 COMPARISON OF STRENGTH DATA FROM CYCLIC TRIAXIAL TESTS ON CHAMPLAIN CLAY-REMOLDED

CHAPTER 6

SUMMARY AND COMMENT

Summary

This report has presented a detailed investigation of the effect of frictionless ends vs regular ends in the cyclic triaxial testing of soils. The study was carried out by running numerous cyclic triaxial tests on a variety of soils under different conditions of density, sample size, end platen conditions, isotropic consolidation stress and cyclic loading frequencies. In addition to the original data generated in this study, previous data on cyclic testing, and the conclusions from static tests reported elsewhere by Lee (2), which were relative to this study, have also been presented.

Prior to a direct study on the effect of end restraint, two basic related questions first had to be answered. The first of these questions dealt with the possible effect of using short prongs in the center of the frictionless cap and base which extended into the soil a short distance to prevent the sample sliding off the ground ends. This question was resolved by running tests for each sand soil studied using regular ends with and without the prongs. The test results indicated that the prongs had no effect on the cyclic strength of these sand soils. The second of these two basic preliminary question had to do with the possible effect of loading frequency on the cyclic strength, because it could be argued that the grease on the frictionless ends would only be effective if the loading were applied slowly. This question was answered by performing tests on sand samples with regular and with frictionless ends at cyclic frequencies of 1.0 Hz

and 0.05 Hz. The results of these tests showed that the frequency at which the tests were run did not effect the cyclic strength of the soil tested. Thus, after answering these questions for each sand soil, comparative tests were run to establish the effect of only end restraint on the cyclic triaxial strength of soils. Nevertheless, in almost all cases, the comparison in strength for regular and frictionless ends was made using ends with prongs and tests at the slow frequency of 0.05 Hz for both regular and frictionless end platens.

The effect of sample size, K_c ratio and sample preparation were also studied incidentally, to a limited extent, for loose Monterey sand. The test data obtained indicated that neither the sample size nor the method of preparation affected the general conclusions regarding the effect of frictionless ends on the cyclic strength. Furthermore, the data also indicated that the same cyclic strengths were obtained whether 1.4 in. diam. or 2.8 in. diam. samples of soil were used for testing.

The main object of this study was to investigate, in a quantitative way, the possible effect of using frictionless end platens on cyclic triaxial test specimens as opposed to using regular platens which developed high frictional resistance against the soil. The individual results for each soil and test case have been summarized at appropriate places throughout the report. For convenient quick reference, the overall results for all soils and cases are summarized in Table 6-1.

With two exceptions, the cyclic strength was increased by the use of frictionless ends as compared with the strength from using regular ends. The two exceptions were a fine silty sand and a clay.

Table 6-1. Summary of Strength Gain Caused by Frictionless Ends as Compared with Regular Ends in Cyclic Triaxial Tests

Case	Soil & Condition	D_{50}	Dr	σ_{3c}	σ_{3crit}	D	Strength Gain (SG) From Frictionless Ends - % For Failure in N cycles shown Below				Ref. Fig.
		mm	%	kg/cm^2	kg/cm^2	kg/cm^2	1-3	10-30	100-300	1000	
1	Monterey No. 0 Moist Tamp K_c = 1.0	0.36	60	0.6	8	7.4	14	12			2-34
2	Monterey No. 0 K_c = 1.0	0.36	60	1.0	8	7	16	15	14	9	2-26
3	Monterey No. 0 K_c = 1.5	0.36	60	1.0	8	7		14	14		2-27
4	Monterey No. 0 K_c = 2.0	0.36	60	1.0	8	7		23	19		2-27
5	Monterey No. 0 K_c = 1.0	0.36	80	1.0	13	12		21	26		2-32
6	Sacramento River K_c = 1.0	0.2	38	1.0	6	5	14	15	14		3-8
7	Sacramento River K_c = 1.0	0.2	80	1.0	10.5	9.5		32	23		3-11
8	Sacramento River K_c = 1.0	0.2	100	1.0	17	16			37		3-12
9	L.A. Harbor K_c = 1.0	0.15	60	1.0	3	2	0	0	0		4-3 4-4
10	Clay (UU)	<0.01	-	-	0 (?)	0 (?)	0	0	0		5-2 5-4

Strength gain = SG = $(S_F - S_R) / S_R$

Dilation = D = $\sigma_{3crit.} - \sigma_{3c}$

σ_{3c} = Minor principal effective consolidation stress

where: S = cyclic strength ($\sigma_{dp}/2\sigma_{3c}$ for specified failure criteria); F = frictionless ends; R = regular ends.

σ_{3crit} = critical effective confining pressure to give zero volume change tendency at failure during static loading

The amount of relative strength increase due to frictionless ends seemed to increase with increasing relative density and with increasing K_c ratio.

Comment

While the foregoing statements adequately summarize the various test results obtained from this study, they do not explain why the data fall into the pattern observed. It is beyond the scope of this report to explain the observed pattern of data in detail. In fact, a detailed explanation is probably not known or not possible to obtain with the data and understanding available at this time. However, a few comments are offered here as a possible aid to explaining the nature of the observed data.

The previous report (2) discussed the effect of soil dilatancy on the strength gain in undrained tests on saturated soils produced by frictionless ends. It was shown that an increasing tendency for dilation led to an increasing strength gain with frictionless platens. It was also suggested that the critical effective confining pressure, σ_{3crit}, provided a convenient measure of dilation tendency. Finally, it was hypothesized that a similar trend might be found for cyclic triaxial tests.

Thus, in following this line of reasoning, it is convenient to relate the observed frictionless end effects as listed in Table 6-1 with the dilation tendencies for the different soils tested as expressed in terms of their critical confining pressures.

The reader is referred to the previous report (2) and the references contained therein for a detailed discussion of the critical confining

pressure for soils. For continuity however, it may be useful to restate here that the critical confining pressure of a soil is defined as that initial consolidation pressure at which a sample will show zero volume change (dilatancy) tendency at failure. Values of σ_{3crit} known to the writers have generally been obtained from static triaxial compression tests on isotropically consolidated ($K_c = 1.0$) samples. Such data may be obtained by interpolation from a series of drained tests with volume change measurements, or from undrained tests on saturated soils with pore pressure measurements. Note that the tendency for volume change varies with soil density so there will be a different value of σ_{3crit} for each density for each soil.

Values of σ_{3crit} for some soils at various relative densities are available from previous studies. Some of the available data which have potential relevance to this study have been plotted in Fig. 6-1. Using the data and trends shown in Fig. 6-1, it was possible to estimate the values of σ_{3crit} for the different soil and density conditions used in the study herein.

In stating that σ_{3crit} is the effective confining pressure at which a soil sample will show zero volume change tendency when sheared at failure, it follows that during an undrained test on a saturated soil, the pore pressures will so adjust that the effective confining pressure at failure will always be σ_{3crit}, no matter what initial effective consolidation pressure σ_{3c} to which the sample was consolidated prior to closing the drainage value and commencing the undrained loading. Thus, a quantitative measure of the dilation tendency for a soil will be

$$D = \sigma_{3crit} - \sigma_{3c} \qquad (6\text{-}1)$$

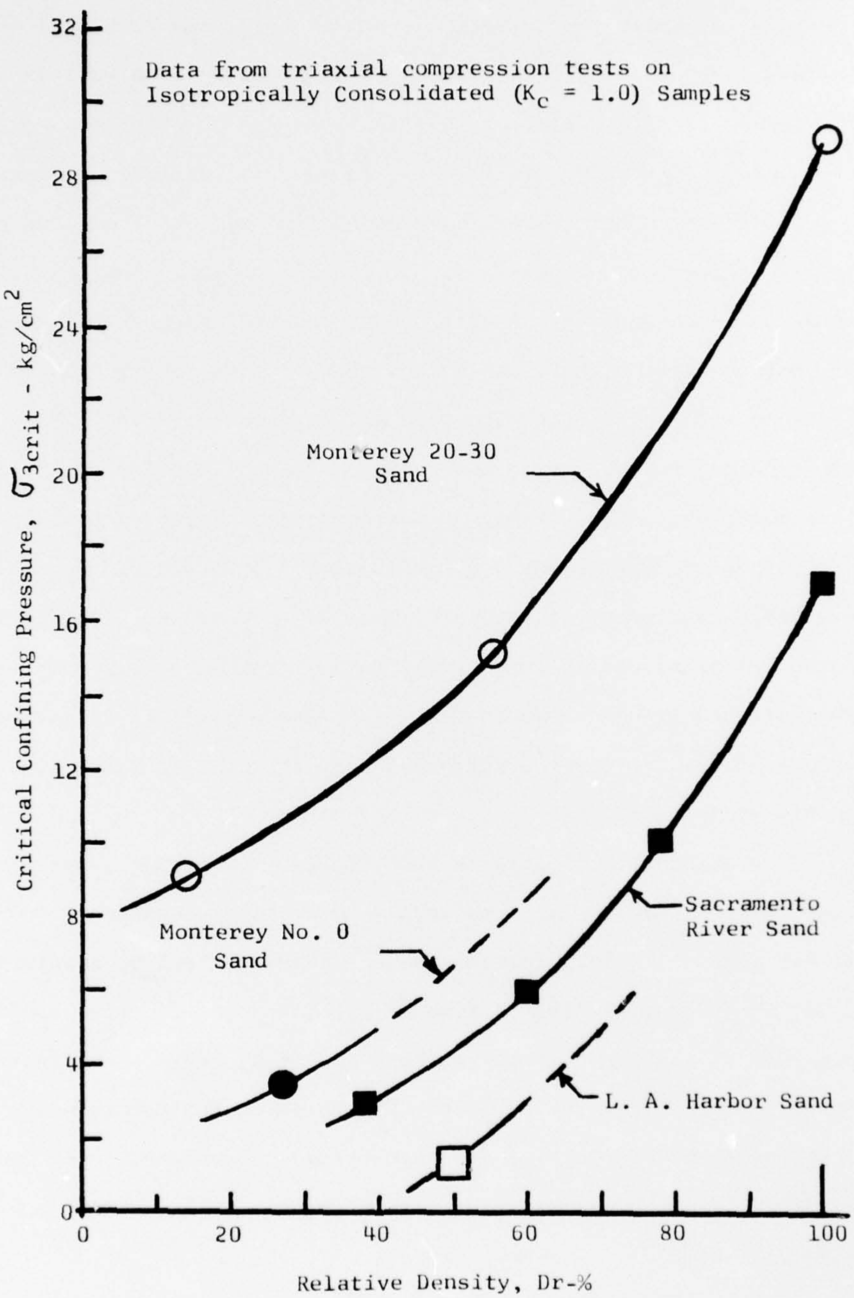

Fig. 6-1 CRITICAL CONFINING PRESSURES FOR SOME SOILS AT DIFFERENT RELATIVE DENSITIES

since the difference D represents the pore pressure (or effective stress) change that must occur to prevent a soil from dilating when sheared. If D is positive, the sense of the dilation tendency is volume expansion when sheared. In an undrained test this would translate into a pore pressure decrease until equilibrium was established at $\sigma_3' = \sigma_{3crit}$. This is the case for most sands, at low consolidation pressure. If D = 0, there will be no volume change or pore pressure change tendency when sheared. If D is negative, the sense of the dilation is a volume decrease in a drained test or a pore pressure increase at failure in an undrained test until equilibrium is established at $\sigma_3' = \sigma_{3crit}$. This is the case for normally consolidated clays, silty sands and other sands when tested at high consolidation pressures.

From the above description of dilation tendency, and the hypothesis suggested in the previous report (2), it might be expected that the strengthening influence of frictionless ends should increase with increasing tendency for dilation, as expressed quantitatively by the factor D. Fig. 6-2 was prepared from Table 6-1 to investigate this hypothesis. As implied by the reasoning described heretofore, the observed data do show a fairly well defined trend of increasing frictionless end effect with increasing soil dilatancy.

The individual soils and conditions are identified in Fig. 6-2 by numbers which refer to Table 6-1. Based on the data presented in this way, the different behavior of the various soils at the different densities has a logical explanation. It is especially interesting that the L. A. Harbor sand, which showed no frictionless end effects should also be only weakly dilatant at the density studied, and therefore the observed cyclic

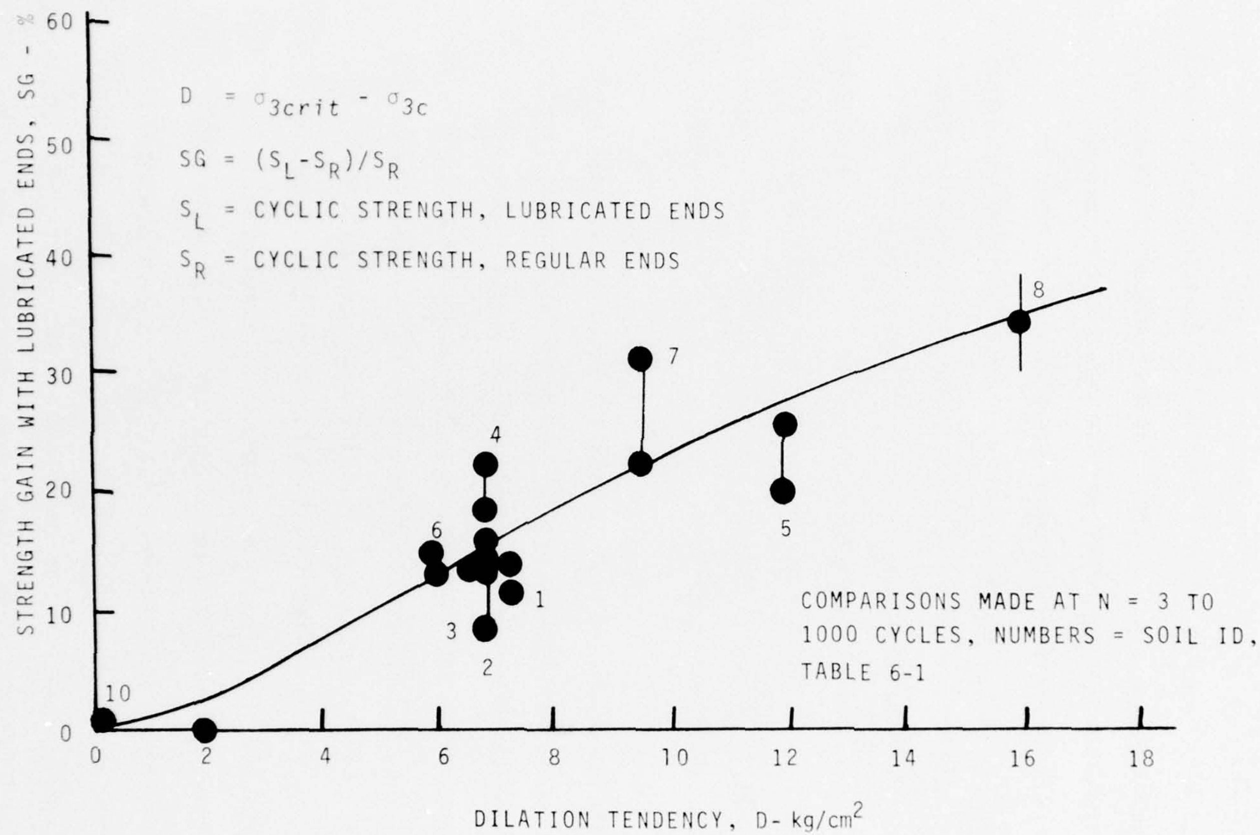

Fig. 6-2 Gain in Strength From Lubricated Ends as a Function of Soil Dilation Tendency

strength behavior is in agreement with the pattern established by the other soils. It is also interesting that the clay soil, which is not dilatant under drained conditions, also showed no changes in strength due to frictionless ends, as should be expected. On the other hand, the very dense Sacramento River sand, which was very dilatant, showed about a 35 percent increase in cyclic strength with frictionless ends as compared with regular ends.

The relative values of increase in strength shown in Table 6-1 and Fig. 6.2, vary depending on the number of cycles at which the comparison was made. Of course, the accuracy of this number depends on the accuracy of the best fit interpretation cruves drawn through the data points, which in turn depends on the number of tests that were made to define each curve.

It is well known that cyclic strength data usually tend to show some scatter. The more data that is obtained, generally the scatter will increase but so will the accuracy of the final interpretative best fit line. Some sets of data in this study involved many tests and some only a few. It is believed that the tests in any series were sufficient in number to define the best fit curves with sufficient accuracy that the overall trends indicated in Table 6-1 and Fig. 6.2 are approximately correct. However, it is also recognized that there is a good possibility of some error in any one set of data so that the band rather than a single line in Fig. 6.2 should be taken as indicative of the trend.

With this limitation in accuracy acknowledged, the apparent discrepancy in the behavior of Monterey No. 0 sand reported by Mulilis (7), using air raining, and that found by this study using both wet raining and moist tamping is probably not very significant.

The Mulilis data shown in Fig. 2-33 does not preclude a frictionless end effect of say 2 or 3 percent, which would be within the band of data if σ_{3crit} for that density and sample preparation method were somewhat lower than used herein. In this regard, it is noted that σ_{3crit} for this sand was obtained by extrapolation from a single data point, and could well be somewhat in error.

All in all, as a final overall statement, it seems reasonable to conclude from the data available that frictionless ends have a potential capacity for increasing the apparent cyclic strength of some soils over that which would be measured using cyclic triaxial tests on specimens with regular end platens. The amount of strength increase appears to be a function of dilation tendency and ranges from as much as 50 percent for very dilatant dense sands down to 10 to 20 percent or less for loose sands. Non-dilatant soils such as clays and loose silty sands show no effect from frictionless ends.

Note that the high effect may be of limited practical interest since few engineering studies would require cyclic liquefaction tests to be performed on very dense sands. The greatest practical interest would be for loose sands for which cyclic triaxial tests are often performed to evaluate the liquefaction potential. For these soils the effect of frictionless ends is small. Furthermore, as described in the previous report (2), the possibility of some small increase of this order of magnitude was already considered by Seed in developing the correction factor C_r relating the results of cyclic triaxial tests to field strengths.

Therefore, as a final comment, the results of this study would appear to be reassuring to those who have in the past, and who wish

in the future to use cyclic triaxial tests with regular end platens for evaluating the liquefaction potential of soil. The data do not suggest a need or an advantage of changing to frictionless ends for routine engineering design purposes provided the data are used with the values of C_r and other technique procedures which have been shown elsewhere to be appropriate for field liquefaction analysis. However, for some basic research or other special purposes, it may be desirable to use frictionless ends in cyclic triaxial tests on some soils.

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This report is based on a Master of Science Thesis by the first named author, Frank J. Vernese, done at the University of California, Los Angeles (UCLA) in 1975. The tests were all performed in the Soil Mechanics Laboratory and most were done by Frank Vernese. However, on occasion he was assisted by two other graduate student research assistants, Claude Corvino and James J. Weaver. The three graduate students worked as a team in studies related to the strength of soil under cyclic loading.

During the period of time when the data for this study was being obtained, Frank Vernese was on leave of absence from and receiving partial support from the firm Dames and Moore in Cranford, New Jersey. In addition, the U.S. Army Corps of Engineers, Waterways Experiment Station (WES) at Vicksburg, Mississippi, also contributed substantially to this study through a research contract to UCLA under the direction of the second named author, K. L. Lee.

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Appendix A

Listing of Significant Data For

Tests Performed in this Study

TABLE A-1

SUMMARY OF DATA FROM TESTS ON MONTEREY NO. SAND

(Dr=60%)

Sample No. (MO-)	γ_{dp} (pcf)	Sample Dia. (in.)	B	Kc	σ_{3c} (psi)	ω (Hz)	σ_{dp} (psi)	$\sigma_{dp}/2\sigma_{3c}$	* Type of End Platen 1,2,3 or 4	Cycles to Failure 5% Single Amp.	Cycles to Failure 5% Double Amp.
1	98.5	1.4	1	1.0	14.5	1.0	6.4	0.22	3	55	48
2	98.5	1.4	1	1.0	14.5	1.0	8.4	0.29	3	10	8
3	98.5	1.4	1	1.0	14.5	1.0	8.7	0.30	4	7	5
4	98.5	1.4	1	1.0	14.5	1.0	7.0	0.24	4	37	35
5	98.5	1.4	1	1.0	14.5	0.06	7.6	0.26	3	14	12
6	98.5	1.4	1	1.0	14.5	0.05	7.0	0.24	3	53	37
7	98.5	1.4	1	1.0	14.5	1.0	7.0	0.24	1	12	12
8	98.5	1.4	1	1.0	14.5	1.0	8.2	0.28	1	3	3
9	98.5	1.4	1	1.0	14.5	0.05	6.6	0.23	1	12	13
10	98.5	1.4	1	1.0	14.5	0.05	8.3	0.28	1	3	4
11	98.5	1.4	1	1.0	14.5	1.0	5.6	0.19	1	60	59
12	98.5	1.4	1	1.0	14.5	0.05	9.7	0.33	3	5	5
13	98.5	1.4	1	1.0	14.5	1.0	9.7	0.33	3	4	4
14	98.5	1.4	(Test not used;piston sheared during test)								
14B	98.5	1.4	1	1.0	14.5	1.0	6.1	0.21	1	15	15
15	98.5	1.4	1	1.0	14.5	1.0	4.6	0.16	1(no failure)		
16	98.5	1.4	1	1.0	14.5	1.0	4.9	0.17	1	970	970
17	98.5	1.4	1	1.0	14.5	1.0	6.0	0.21	3	72	64
18	98.5	1.4	(Test not used;leak during test)								
19	98.5	1.4	1	1.0	14.5	1.0	5.5	0.19	3	137	140
20	98.5	1.4	1	1.0	14.7	0.05	11.6	0.40	3	2	2
21	98.5	1.4	1	1.0	14.5	1.0	10.8	0.37	1	1	1
22	98.5	1.4	1	1.0	14.5	0.05	5.2	0.18	3	397	408
23	98.5	2.8	1	1.5	14.1	0.05	8.6	0.30	2	42	***
24	98.5	2.8	1	1.5	14.5	0.05	11.6	0.40	2	7	***
25	98.5	2.8	1	1.5	14.5	0.05	7.6	0.26	2	285	***
26	98.5	2.8	1	1.5	14.5	0.05	11.6	0.40	3	12	***
27	98.5	2.8	1	1.5	14.5	0.05	8.9	0.31	3	89	***
28	98.5	2.8	(Test not used; leak during test)								
29	98.5	2.8	1	2.0	14.5	0.05	16.0	0.50	2	156	***
30	98.5	2.8	1	2.0	14.5	0.05	18.5	0.63	2	11	***
31	98.5	2.8	1	2.0	14.5	0.05	16.6	0.54	2	50	***
32	98.5	2.8	1	2.0	14.5	0.05	18.5	0.63	3	101	***
33	98.5	2.8	1	2.0	14.5	0.05	16.5	0.57	3	391	***
34	98.5	2.8	1	2.0	14.5	0.05	17.0	0.55	3	620	***
35	98.5	2.8	1	1.5	14.5	0.05	14.0	0.48	3	6	***
36	98.5	2.8	1	1.0	14.5	0.05	14.5	0.19	3	189	***
37	98.5	2.8	1	1.0	14.5	0.05	8.2	0.28	2	6	5
38	98.5	2.8	(Test not used; leak in membrane)								

TABLE A-1 (CONTINUED)

Sample No. (MO-	γ_d (pcf)	Sample Dia. (in.)	B	Kc	σ_{3c} (psi)	ω (Hz)	σ_{dp} (psi)	$\sigma_{dp}/2\sigma_{3c}$	Type of End Platen 1,2,3,or 4 *	Cycles to Failure,5% Single Amp.	Cycles to Failure,5% Double Amp
39	98.5	2.8	1	1.0	14.5	0.05	6.1	0.21	3	41	41
40	98.5	2.8	1	1.0	14.5	0.05	8.1	0.28	3	9	8
41	98.5	2.8	1	1.0	14.5	0.05	5.7	0.20	2	25	24
42	98.5	2.8	1	1.0	14.5	0.05	9.5	0.22	2	4	3
43	98.5	2.8	1	1.0	14.5	0.05	11.5	0.36	3	5	4
					(Dr=90%)						
44	105.0	2.8	1	1.0	14.5	0.05	11.6	0.40	2	59	59
45	105.0	2.8	1	1.0	14.5	0.05	12.4	0.44	2	140	121
46	105.0	2.8	1	1.0	14.5	0.05	15.0	0.52	2	***	165
47	105.0	2.8	1	1.0	14.5	0.05	20.0	0.69	2	***	220
48	105.0	2.8	1	1.0	14.5	0.05	20.0	0.69	3	116	116
49	105.0	2.8	1	1.0	14.5	0.05	25.0	0.86	3	33	33
50	105.0	2.8	1	1.0	14.5	0.05	25.0	0.86	2	108	108
51	105.0	2.8	1	1.0	14.5	0.05	15.0	0.52	3	85	85
					(Dr=80%)						
52	103.5	2.8	(Test		not used;		leak during		test)		
53	103.5	2.8	1	1.0	14.5	0.05	15.0	0.52	3	24	24
54	103.5	2.8	1	1.0	14.5	0.05	15.4	0.58	2	30	30
55	103.5	2.8	1	1.0	14.5	0.05	11.4	0.35	2	24	24
56	103.5	2.8	1	1.0	14.5	0.05	5.4	0.17	2	298	298
57	103.5	2.8	1	1.0	14.5	0.05	8.1	0.28	3	125	125
58	103.5	2.8	1	1.0	14.5	0.05	9.4	0.21	3	169	169
59	103.5	2.8	1	1.0	14.5	0.05	11.4	0.35	3	34	34
60	103.5	2.8	1	1.0	14.5	0.05	13.9	0.47	2	14	14
61	103.5	2.8	1	1.0	14.5	0.05	10.1	0.24	2	62	62
62	103.5	2.8	1	1.0	14.5	0.05	14.0	0.48	3	30	30
					(Dr=60%)						
63**	98.5	2.8	(Test		not used;		leak during		test)		
64**	98.5	2.8	1	1.0	8.0	0.05	6.4	0.39	2	20	20
65**	98.5	2.8	1	1.0	8.0	0.05	4.8	0.29	2	13	13
66**	98.5	2.8	1	1.0	8.0	0.05	3.9	0.24	2	15	15
67**	98.5	2.8	1	1.0	8.0	0.05	3.9	0.24	3	64	64
68**	98.5	2.8	1	1.0	8.0	0.05	5.4	0.34	3	6	6
69**	98.5	2.8	1	1.0	8.0	0.05	4.9	0.31	3	10	10

Note: *1=Regular Ends, 2=Regular Ends with Prongs,
3=Frictionless Ends - 2 layers of grease,
4=Frictionless Ends - 1 layer of grease.
**Samples prepared by tamping in six layers.
***Samples did not reach failure criteria.

TABLE A - 2

SUMMARY OF DATA FROM TESTS ON SACRAMENTO RIVER SAND

Sample No.** (-)	γ_d (pcf)	Sample Dia. (in.)	B	Kc	σ_{3c} (psi)	ω (Hz)	σ_{dp} (psi)	$\sigma_{dp}/2\sigma_{3c}$	Type of End Platen 1,2,3,4.*	Cycles to Failure 5% Single Amp.	Cycles to Failure 5% Double Amp.
					(Dr=38%)						
11	89.43	1.4	1	1.0	15.0	1.0	6.6	0.22	1	6	6
12	89.43	1.4	1	1.0	15.0	1.0	5.4	0.18	1	26	26
13	89.43	1.4	1	1.0	15.0	1.0	7.2	0.24	1	5	5
14	89.43	1.4	1	1.0	15.0	1.0	6.0	0.20	1	12	11
15	89.43	1.4	1	1.0	15.0	1.0	6.0	0.20	2	14	14
16	89.43	1.4	1	1.0	15.0	1.0	6.3	0.21	2	12	12
17	89.43	1.4	1	1.0	15.0	1.0	7.8	0.26	2	2.5	2.5
18	89.43	1.4	1	1.0	15.0	1.0	4.8	0.16	2	47	47
19	89.43	1.4	1	1.0	15.0	1.0	4.8	0.16	3	147	140
20	89.43	1.4	1	1.0	15.0	1.0	6.3	0.21	3	19	18
21	89.43	1.4	1	1.0	15.0	1.0	7.8	0.26	3	5	5
22	89.43	1.4	1	1.0	15.0	1.0	8.6	0.29	3	5	8
23	89.43	1.4	1	1.0	14.5	0.05	7.3	0.25	3	8	8
24	89.43	1.4	1	1.0	15.0	1.0	5.7	0.19	3	23	23
25	89.43	1.4	1	1.0	15.0	0.05	5.7	0.19	3	17	17
26	89.43	1.4	1	1.0	15.0	0.05	4.5	0.15	3	46	46
					(Dr=78%)						
27	97.8	1.4	1	1.0	15.0	0.05	8.7	0.29	2	No Failure	
28	97.8	1.4	1	1.0	15.0	0.05	9.0	0.30	2	10	12
29	97.8	1.4	1	1.0	15.0	0.05	11.7	0.39	1	12	20
30	97.8	1.4	1	1.0	15.0	0.05	14.7	0.49	2	5	15
31	97.8	1.4	1	1.0	15.0	0.05	9.9	0.33	2	23	23
32	97.8	1.4	1	1.0	15.0	1.0	9.6	0.32	3	204	108
33	97.8	1.4	1	1.0	15.0	1.0	11.4	0.38	3	279	79
34	97.8	1.4	1	1.0	15.0	1.0	14.7	0.49	3	15	15
35	97.8	1.4	1	1.0	15.0	1.0	12.9	0.43	3	37	32
36	97.8	1.4	1	1.0	15.0	1.0	12.2	0.41	1	8	12
37	97.8	1.4	1	1.0	15.0	0.05	14.8	0.49	3	12	17
38	97.8	1.4	1	1.0	15.0	1.0	14.8	0.49	1	5	13
39	(Test not used, sample prepared to dense)										
40	97.8	1.4	1	1.0	15.0	0.05	10.0	0.33	1	15	15
41	97.8	1.4	1	1.0	15.0	1.0	9.0	0.30	1	225	85

Note: *1=Regular Ends, 2=Regular Ends with Prongs,
3=Frictionless Ends - 2 layers of grease,
4=Frictionless Ends - 1 layer of grease.

**Samples 1 - 10 were used to familiarize the writer with testing procedure and therefore are not presented.

TABLE A-3

SUMMARY OF DATA FROM TESTS ON L.A. HARBOR SAND

Sample No. (1A-	(pcf)	Sample Dia. (in.)	B	Kc	(psi)	(Hz)	dp (psi)		Type of End Platen 1,2,3,4.	Cycles to * Failure 5% Single Amp.	Cycles to Failure 5% Double Amp.
1	95.0	2.8	1	1.0	15.0	1.0	8.1	0.27	3	7	6
2	95.0	2.8	1	1.0	15.0	1.0	7.5	0.25	3	7	6
3	95.0	2.8	1	1.0	15.0	1.0	6.4	0.21	3	13	12
4A	95.0	2.8	1	1.0	15.0	1.0	8.8	0.29	1	3	3
4B	95.0	2.8	1	1.0	15.0	1.0	5.2	0.17	1	25	25
5	95.0	2.8	1	1.0	15.0	1.0	5.2	0.17	2	90	90
6	95.0	2.8	1	1.0	15.0	1.0	7.2	0.24	2	12	12
7	95.0	2.8	(Test not used, leak durinq test)								
8	95.0	2.8	1	1.0	15.0	0.05	9.4	0.315	2	3	3
9	95.0	2.8	1	1.0	15.0	0.05	7.0	0.23	2	26	26
10	95.0	2.8	(Test not used, leak during test)								
11	95.0	2.8	1	1.0	15.0	0.05	9.2	0.31	3	3	3
12	95.0	2.8	(Test not used, leak during test)								
13	95.0	2.8	1	1.0	15.0	0.05	6.5	0.217	2	15	15
14	95.0	2.8	1	1.0	15.0	0.05	6.2	0.207	2	15	15
15	95.0	2.8	1	1.0	15.0	0.05	4.5	0.150	3	85	85

Note: *1=Regular Ends, 2=Regular Ends with Prongs,
3=Frictionless Ends - 2 layers of grease,
4=Frictionless Ends - 1 layer of grease

TABLE A-4

SUMMARY OF DATA FROM TESTS ON CHAMPLAIN CLAY

Sample No. (HQ-	γ_d (pcf)	Sample Dia. (in.)	Kc	Water Content (%)	σ_3 (psi)	ω (Hz)	Sample Htg. (in.)	σ_{dp}	Type of * End Platen 1,2,or3.	Cycles to Failure 3% Single Amp.
				UNDISTURBED						
1	104.6	1.4	1	23	60	1.0	2.9	30	1	39
2	103.8	1.4	1	22	60	1.0	3.0	30	3	39
3	103.6	1.4	1	21	60	1.0	3.1	40	3	15
4	102.8	1.4	1	23	60	1.0	3.0	40	1	15
5	104.8	(Test not used; Prongs caused failure)								
6	104.2	(Test not used; Prongs caused failure)								
				REMOLDED						
1	117.0	1.4	1	15	30	1.0	3.4	25	1	no failure
2	117.0	1.4	1	15	30	1.0	3.4	48	3	1
3	117.0	1.4	1	15	30	1.0	3.4	39	3	67
4	117.0	1.4	1	15	30	1.0	3.4	41	1	3
5	117.0	1.4	1	15	30	1.0	3.4	39	1	62
6	117.0	1.4	1	15	30	1.0	3.4	36	3	163

Note: * 1=Regular Ends, 2=Regular Ends with Prongs, 3=Frictionless Ends without Prongs,

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